DESIGN OF A DEEP SEWERAGE SYSTEM

FOR THE

CITY OF WHITING, INDIANA

BY

CHARLES EDMUND DE LEUW

B. S. UNIVERSITY OF ILLINOIS, 1912

THESIS

SUBMITTED IN PARTIAL FULFILLMENT
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I HEREBY RECOMMEND THAT THE THESIS PREPARED BY

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ENTITLED DESIGN OF A DEEP SEWERAGE SYSTEM FOR THE CITY OF WHITING, INDIANA.

BE ACCEPTED AS FULFILLING THIS PART ON THE REQUIREMENTS FOR THE PROFESSIONAL DEGREE OF Civil Engineer

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DESIGN OF A DEEP SEWERAGE SYSTEM FOR THE CITY OF WHITING, INDIANA.

CHAPTER VIII.

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DESIGN OF A DEEP SEWERAGE SYSTEM FOR THE CITY OF WHITING, INDIANA.

CHAPTER I.

INTRODUCTION.

(A) Necessity for Deep Sewerage System.

A portion of the municipal engineer's work, at the present time, consists in the rehabilitation of existing sewerage systems or the construction of auxiliary and relief sewerage systems. This is the result of various causes, which will be briefly related.

It is not surprising, that in the early days of the art of sewer design, competent engineers made mistakes in design which resulted in the works proving themselves inadequate, because the design of sewerage systems is an inexact problem even now, when we have the advantage of years of study, and masses of experimental data. It was not always the case that the construction of sewerage works was in charge of competent engineers, and many systems proved inadequate even at the time of their construction, due to the fact that the men responsible for their design had no idea of the fundamental principles involved in the design of a sewerage system.

In many cases sewers were made only large enough to care for the existing population and conditions, due to financial considerations. It is a recognized fact that there is a limit to the length of time for which we must provide in the future, the determination of which is a problem in economics. American municipalities have grown very rapidly as a whole, and in individual cases their growth has exceeded all expectation. This growth has been both in area and in population, consequently it has sometimes been necessary to redesign their public works to keep pace with their growth.

The City of Whiting, Indiana, is in need of an auxiliary sewerage system at the present time, the city having grown in population and in area since the first system of sewers was constructed. The system, which is proposed now, would probably have been impossible from a financial viewpoint, at the time of the construction of the existing system.

This city is now served by a system of combined sanitary and storm sewers in that portion of the city lying north of 125th Street. That portion lying south of 125th Street is not provided with sewers, and as the existing sewerage system is on too high a level to serve this district, some means of providing relief for this south district must be devised.

There has been a demand by property owners in the business district of the city for drainage of their basements. The existing sewerage system has not sufficient depth to adequately drain the basements under the stores, therefore it is necessary to construct a system of deep sewers to provide sufficient drainage for this district.

The existing high level sewers provide adequate storm water drainage for all that portion of the city which they serve, but they do not adequately drain the basements, and any deep sewerage system must be so designed that it will provide for properly carrying away the sanitary sewage of the entire city.

(B) Topography of Whiting.

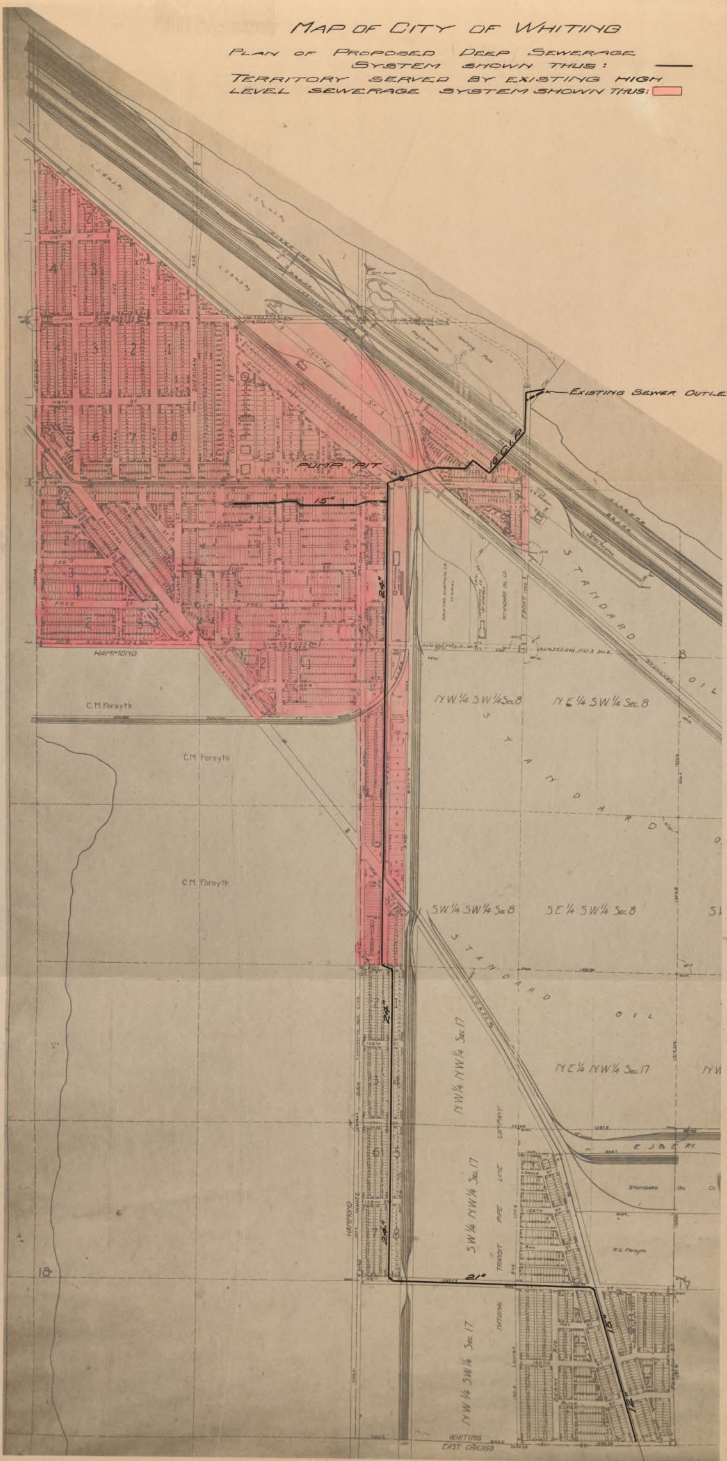
Whiting, Indiana, is situated on the southwestern shore of Lake Michigan, about seventeen miles southeast of Chicago. The country is practically level, the elevations varying between four (4) and eight (8) feet above lake level. The soil is composed almost altogether of fine sand, known locally as lake sand, with a very small proportion of loam on the surface. This sand may be described as open sand and has a high degree of porosity. The ground-water plane in a bed of this sand has almost no slope, due to the fact that the water flows freely through the sand. The sand extends to a depth of over twenty five (25) feet below the surface of the ground.

Most of the streets in that portion of the city north of 126th Street are improved with pavements. On the principal streets brick has been used, and on others, bituminous and water bound macadam. The grade of most of the improved streets has been raised to an elevation of about six (6) feet above lake level, the business streets being a little higher. By reference to the map, it will be seen that a considerable portion of the city is occupied by the refineries of the Standard Oil Company and as they provide their own sewage works, no further consideration will be given that territory.

The population of Whiting was 6,587 in 1910. That portion north of 125th Street is well built up, and the population is dense. The portion south of 125th Street is very sparsely settled, the entire population of this district being about 500. The main business district is along 119th Street, between Sheridan Avenue and Front Street.

(C) Existing Sewerage System.

The existing sewerage system serves that portion of the city shown on the map in light red as a combined sanitary and storm system. The outlet is a three (3) foot circular sewer and the grade of the invert at the outlet is at the lake level. The outlet is located as shown on the map. While this system of sewers has proven adequate for storm water drainage, the grades are high


MAP OF CITY OF WHITING
PLAN OF PROPOSED DEEP SEWERAGE SYSTEM SHOWN THUS:
TERRITORY SERVED BY EXISTING HIGH LEVEL SEWERAGE SYSTEM SHOWN THUS:
EXISTING SEWER OUTLET
PUMP PIT
15"
24"
21"
18"
HAMMOND
C.M. Forsyth
C.M. Forsyth
C.M. Forsyth
NW 1/4 SW 1/4 Sec 8
N.E 1/4 SW 1/4 Sec 8
SW 1/4 SW 1/4 Sec 8
SE 1/4 SW 1/4 Sec 8
NW 1/4 NW 1/4 Sec 17
N.E 1/4 NW 1/4 Sec 17
SW 1/4 NW 1/4 Sec 17
NW 1/4 SW 1/4 Sec 17
STANDARD OIL
E J & E RY
WHITING
EAST CHICAGO

in the residence district, and the basements are shallow and poorly drained.

(D) Synopsis.

The writer will discuss the general principles involved in sewer design. A main sanitary and storm sewer will be designed for that portion of the city south of 126th Street. This sewer will start at 131st Street and Indiana Boulevard, thence northerly in Indiana Boulevard to 129th Street, thence west in 129th Street to Schrage Avenue, thence north in Schrage Avenue to 126th Street. An approximate design of a sanitary sewerage system will be made for the district north of 126th Street, and a main sanitary sewer will be designed which will start in the alley south of 119th Street at the center line of Oliver Street extended and run thence to a connection with the sewer in Schrage Avenue. A sewer will be designed to receive the flow of the above sewers and will commence at 126th Street and Schrage Avenue, thence north in Schrage Avenue to the pumping station, located on the northeast corner of Schrage Avenue and 119th Street.

A pumping station and an outlet sewer from the pumping station to the lake will be designed.

It is, perhaps, needless to state that the location of these works has not been selected arbitrarily. A thorough preliminary study of the various means of accomplishing the ends desired was made. An estimate of cost of other possible systems was made, and this location choosen because of economical reasons. This will be discussed briefly in Chapter 6, but for fear of going to a tiresome length, the writer will not discuss this phase of the problem in detail.

An estimate of cost of the sewers designed will be made, and on account of the peculiar conditions obtaining in this locality, the construction methods and cost data available will be given in detail. The plans and the important portions of the specifications will be given in conclusion.

CHAPTER II.

GENERAL PRINCIPLES OF SEWER DESIGN.

(A) Flow in Sewers.

Sewage is a liquid which has qualities very similar to water. The specific gravity of sewage has been found to be a little greater than that of water; but for all practical purposes, sewage may be treated as water in the discussion of hydraulics.

The Chezy formula for velocity of flow in sewers has been commonly used by engineers for years. It is $v = c\sqrt{R.S.}$ in which v is the mean velocity, c a constant, R the hydraulic radius or ratio of the area of cross-section to the wetted perimeter, and S the sine of the slope. It is based upon the following facts. The flow in sewers is due to the force of gravity. The formula for the velocity of a freely falling body is $v = \sqrt{2\ g\ h}$ where v = velocity in feet per second, g is the acceleration due to gravity, about 32 feet per second, and h the head in feet. In the case of water falling in sewers there is a retarding force due to the friction on the sides of the sewers. The friction increased as the length increases, therefore the velocity varies inversely as some function of the length l. The friction also decreases as the hydraulic radius increases, hence the velocity varies directly as some function of the hydraulic radius.

Combining these facts and expressing them as a formula, we have $v = \frac{c\sqrt{2\ g\ h}\ f\ (R)}{f\ (l)}$ where c is a constant indicative of the roughness of the surface. As the head, h, in a sewer is the total fall from the point of beginning to any point under consideration, and the length, l, is the total length of the same, it follows that $\frac{h}{l}$ is the sine of the slope. This may be written $v = c\sqrt{R\ \frac{h}{l}}$ in which c includes $\sqrt{2\ g}$, and in the terms of the Chezy formula

$v = c\sqrt{R\,S}$. Any formula for c is essentially empirical, the one is most common usage being Ganguillet and Kutter's, which is:

$$c = \frac{41.66 + \frac{1.811}{n} + \frac{0.00281}{S}}{1 + \left(41.66 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}$$

in which n is the coefficient of roughness of the wetted perimeter, and the other symbols the same as in the Chezy formula.

"It will be seen that c varies with the roughness of the wetted perimeter decreasing with the increase in the roughness, most rapidly when R is small; 2. It varies with the value of the hydraulic radius R, increasing with its increase most rapidly when R is small; 3. It varies with the slope S, decreasing with its increase in large streams and increasing with its increase in small streams;"from Ganguillet and Kutter's Flow of Water in Rivers and other Channels.

The determination of the coefficient of roughness, n, has been made experimentally by many engineers and an outline of some of the results obtained is given below. Ganguillet and Kutter suggested the following values of n.

1. Channels lined with carefully planed boards or with smooth cement, 0.010.

2. Channels lined with common boards, 0.012.

3. Channels lined with neatly jointed brick work, 0.013.

4. Channels in rubble masonry, 0.017.

5. Channels in earth, brooks and rivers, 0.025.

6. Streams with detritus or aquatic plants, 0.030.

Louis D. A. Jackson submitted the following suggestions as to the extensions of range from Ganguillet and Kutter's value of n, as follows:

Smooth cement, worked plaster, planed wood and glazed surface in perfect order, n = 0.010. The materials above mentioned under 0.010 when in inferior condition, also brick work, ashlar and unglazed stoneware in a good condition, n = 0.013.

Brick work, ashlar and stoneware in an inferior condition, rubble in cement or plaster in good order n = 0.017. Rubble in cement in an inferior condition. Coarse rubble rough-set in a normal condition, n, = 0.020.

Coarse dry-set rubble in bad condition, n = 0.025.

Theodore Horton in an article upon "Flow in Sewers of the North Metropolitan Sewerage System of Massachusetts", (Transactions American Society of Civil Engineers, 1901, Page 78) gives an account of the guagings made in the Metropolitan sewers. He found a value of n for flow in a large sand washed brick sewer to vary between 0.0115 and 0.0152. He found that the greatest change in the surface of the sewers took place soon after the sewers were put into operation and that the values of n obtained immediately after the sewers were put in operation, increased until the sides of the sewers had their maximum coating of slime.

The Bureau of Surveys at Philadelphia, had a series of observations made in 1909 of the values of the coefficient of roughness, n, of some of the large sewers in Philadelphia, with the following results:

	n
Old sewers, brick bottom not clean	0.017
Old sewers, stone block bottom clean	0.017
New sewers, stone block bottom clean	0.016
New sewers, brick bottom clean	0.015
Concrete or brick sewer, vitrified shale brick invert, clean	0.012 to 0.013
Concrete sewers, granolithic finished bottom	0.011
Open channel box, planed planks	0.011
Old sewers, bad or dirty bottoms	0.017 to 0.020

John Ericson in an article "Investigations of the Flow in Brick-lined Conduits" (Journal Western Society of Engineers, Oct. 1911, p. 657) gives values of n, for several sections in the Northwest land and lake tunnels at Chicago, as follows: 0.01455, 0.01347, 0.01552, 0.01493, 0.01382, and 0.01385; Average =0.01435.

"The author (Ericson) from his experience with these as well as other similar cases, is of the opinion that for tunnels or sewers of ordinary sizes and velocities of flow, lined with sewer brick laid in cement mortar, if the brick is properly selected and not too warped or uneven, laid in a workmanlike manner and true to line, not disturbed by blasting, and the mortar joints scraped off flush with the brick, a coefficient of roughness, n, in Kutter's formula of 0.0130 is readily attainable, if the extraordinary resistances to flow, such as bends, enlargements, etc., are eliminated."

Metcalf and Eddy suggest the following values of coefficient of roughness, n, in the case of sewer pipes, conduits and channels, under reasonably good operating conditions,

	n
For vitrified pipe sewers	0.015
For concrete sewers of large section and best work laid on slopes giving velocities of 3 ft. per second or more	0.012
For concrete sewers under good ordinary conditions of work	0.013
For brick sewers lined with vitrified or reasonably smooth hard burned brick and laid with great care, with close joints	0.014
For brick sewers under ordinary conditions	0.015
For brick sewers laid on flat grade and rough work	0.017 to 0.020

T. Chalkley Hatton gives the results of experiments on the flow of water in two 24-inch sewers built with 3-foot lengths of pipe and with cement joints, at Carlisle, Pa. Experiments on a section 4660 feet long having a grade of 0.077 per cent, and having bends at five manholes with depths of water of 5 and 12 inches, gave n = 0.0128 and n = 0.0112, respectively. One experiment on another section 2095 feet long and having one bend at a manhole and a grade of 0.04 per cent, gave with a depth of 12 inches,

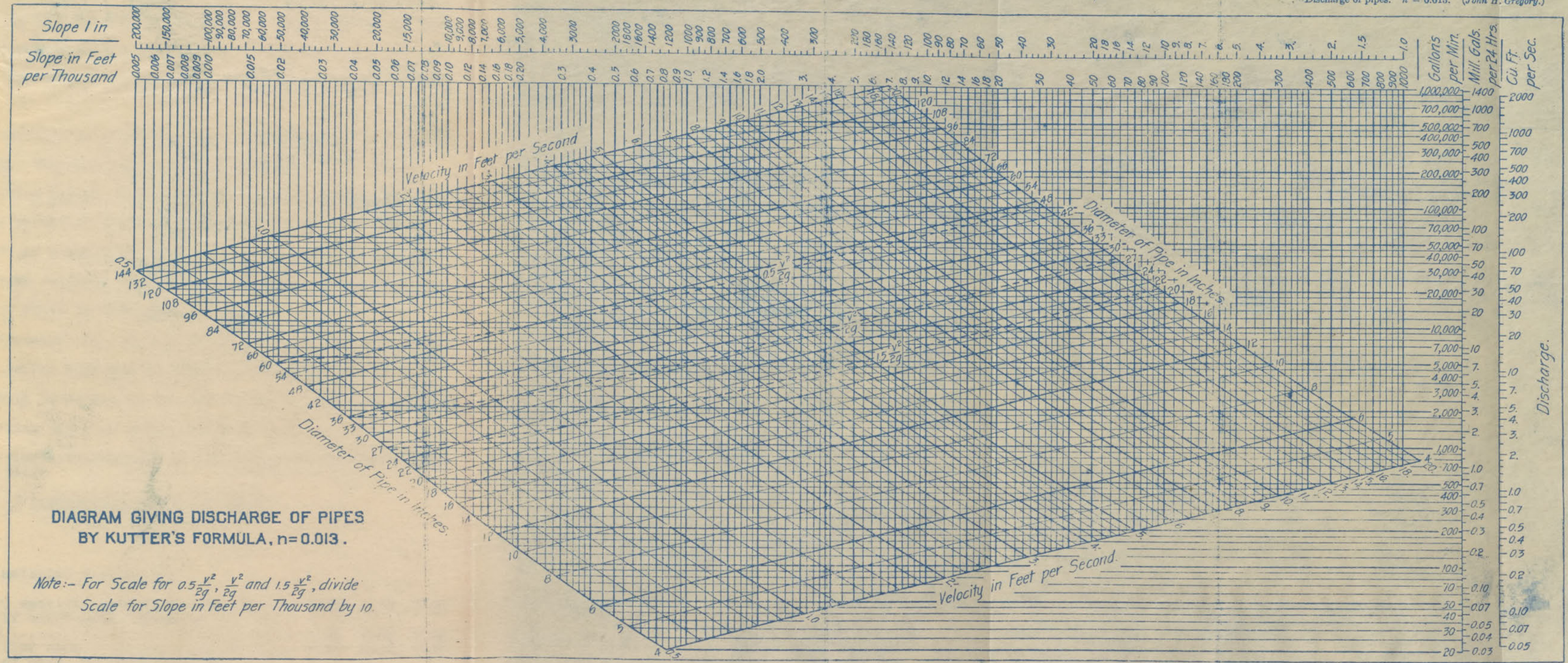


DIAGRAM GIVING DISCHARGE OF PIPES BY KUTTER'S FORMULA, n=0.013.

Note:– For Scale for $0.5\frac{v^2}{2g}$, $\frac{v^2}{2g}$ and $1.5\frac{v^2}{2g}$, divide Scale for Slope in Feet per Thousand by 10.

n = 0.0111.

It is well to remember that Ganguillet and Kutter's formula is entirely empirical and is based only upon experimental data. It has been made valuable for use by means of labor saving charts first prepared in 1901 by John H. Gregory. A reproduction of this diagram for the values of n = 0.013 is given herewith and will be used hereafter in working out the size, velocity and grade of sewers.

The velocity of flow in sewers is subject to the following limitations. The velocity should be such that there is no scouring or wearing action on the surface of the channels but should be large enough so that the sewer will be self cleansing and any sediment or slime will be removed at the time of maximum flow. The minimum velocity should be large enough to prevent any accumulations of filth in the sewer.

The transporting power of water has been the subject of considerable research work and in order to determine the proper velocities it will be necessary for us to have at hand some data as to the transporting power of water, which is given below: (Hering & Trautwine's Translation of Ganguillet and Kutter's "Flow of Water", p. 124).

<u>Transporting Power of Water.</u>

Nature of material forming bed	Velocity – Required to Move ft per sec		
River mud, clay, specific gravity = 2.64	0.25	0.33	0.40
Sand, the size of anise-seed, specific gravity = 2.55	0.35	0.46	0.55
Clay, loam and fine sand	0.50	0.66	0.79
Sand, the size of peas, specific gravity = 2.55	0.60	0.79	0.95
Common river sand, specific gravity = 3.36	0.70	0.92	1.10
Sand, the size of beans, specific gravity = 2.55	1.07	1.40	1.69

Gravel	2.00	2.62	3.15
Round pebbles, one inch diam., specific gravity = 2.61	2.13	2.79	3.36
Coarse gravel, small cobblestones	3.00	3.93	4.73
Angular stones, flint, egg size, specific gravity = 2.25	3.23	4.23	5.09
Angular broken stone	4.00	5.24	6.30
Soft slate, shingle	5.00	6.55	7.86
Stratified rock	6.00	7.86	9.43
Hard rock	10.00	13.12	15.75

Blackwell showed by experiments made for the British Metropolitan Drainage Commission that the specific gravity has a marked effect upon the velocities necessary to move bodies, as given in the following table:

Nature of bodies	Specific gravity	Velocity in feet per second
Coal	1.26	1.25 to 1.50
Coal	1.33	1.50 to 1.75
Brickbat	2.00	1.75 to 2.00
Brickbat	2.12	2.00 to 2.25
Brickbat	2.18	2.25 to 2.50
Piece of flint	2.66	2.50 to 2.75

The Metropolitan Sewerage Commission of New York, 1910, assumed the velocities given in the following table to be necessary to move solid particles:

Kind of material	Velocity required to move on bottom. Feet per second.
Fine clay and silt	0.25
Fine sand	0.50
Pebbles half inch in diameter	1.0
Pebbles 1 inch in diameter	2.0

The scouring effect of sewage upon the invert and sides of sewers depends to a large extent upon the amount of sand or grit admitted to the sewers. It has been found in the cases of large storm sewers that several inches had been worn away in the course of fifteen or twenty years, by velocities ranging from four to ten feet per second. By use of durable linings high velocities may be used on trunk sewers, but on vitrified pipe sewers ~~it is considered~~

it is considered good practice to limit the maximum velocity of flow to about $3\frac{1}{2}$ feet per second.

The flow in sewers varies greatly at different times of the day and at certain times of the day in a sanitary sewer there will be practically no flow at all. At such times it is apparent that the small flow will be sufficient to carry only the very finely suspended particles and will leave coarse particles along the sides of the sewer. Thought must be given to this fact to the end that at necessary intervals of time, sufficient flow and velocity will obtain, that the sewers will be cleansed of all such detritus. In considering this fact, it should be borne in mind that the value of R will be as great when a sewer is flowing half full as when it is flowing full.

It has been the custom of some engineers to design the small size of pipe sewers as flowing half full at their maximum conditions of flow. This has been done to enable them to use larger pipes and small slopes.

The problem of determining minimum and maximum grades is related with the problem of determining size and velocity. The minimum grades for different size sewers adopted in general are such as to produce velocity within the limits above stated.

The minimum grades used on the New Orleans sewers are given in the following table:

Diameter, inches	Slope, per cent.	Diameter, inches	Slope, per cent.	Diameter, inches	Slope, per cent.
8	0.33	27	0.100	48	0.062
10	0.25	30	0.091	51	0.059
12	0.21	33	0.083	54	0.056
15	0.167	36	0.083	57	0.053
18	0.133	39	0.077	60	0.050
21	0.144	42	0.071	63	0.050
24	0.100	45	0.067	66	0.050

Mr. C. E. Grunsky gives the following minimum grades in California Cities:

Sizes, inches	Stockton, per cent.	Fresno, per cent.	Modesto, per cent.	Visalia, per cent.	Sacramento, per cent.
6	0.2	0.15	0.16	0.3	0.25
8	0.143	0.1	0.16	0.24	0.2
10	0.139	0.1	0.2	0.143	0.16
12	0.1	0.1	0.152	0.143	0.12
15			0.09		
18				0.1	

T. Chalkley Hatton in experiments with two 24 inch sewers discharging creek water carrying considerable clay, the grade being 0.077 per cent, found no appreciable sediment with the following depths in inches and velocities in feet per second:

Depth	5	12	12
Velocity	1.21	2.35	1.70

It may be said that it is considered good practice by most engineers to lay vitrified pipe sewers on such grades that the velocity, when flowing full or half full, will be between two and three feet per second.

(B) Quantity of Sewage.

A proper knowledge of the quantity of sewage to be provided for is of fundamental importance in sewer design. There are three different sources of sewage; first, storm water or rainfall flowing to the sewers from the surface of the ground; second, sanitary sewage which carries the waste of house and industrial plants; and third, infiltration or leakage into the sewers of ground water.

In the consideration of the quantity of storm water, it is necessary first to estimate the amount and intensity of rainfall, which it will be necessary to design for. In the average small municipality no data as to rainfall will be at hand. The

United States Weather Bureau has been keeping rainfall records for all important cities in the country, and these records are very valuable if they are close to, and have the same climatic conditions as the locality under consideration. A tabulation of the most important rainfalls in Chicago, from U. S. W. B. Observations, is given below:

Excessive Precipitation, Chicago, Ill.

Year	Date	Depth Inches	Time Hr.	Time Min.	Year	Date	Depth Inches	Time Hr.	Time Min.
1896	May 25	1.32	0	40	1905	Aug. 5	1.20	0	25
1896	Aug. 6	1.17	0	40	1905	Sept.1	1.47	1	00
1897	July 4	0.37	0	15	1905	Sept.18	0.82	0	25
1900	July 2	0.76	1	00	1906	Apr. 28	0.30	0	10
1900	Aug. 17	0.61	0	25	1906	July 15	1.26	0	30
1901	June 5	0.88	0	30	1907	July 11	0.55	0	25
1901	July 1	1.07	0	30	1907	July 22	0.52	0	30
1902	May 24	1.08	0	45	1907	Aug. 16	1.00	1	00
1902	June 7	0.58	0	35	1907	Aug. 27	0.72	0	20
1902	July 23	0.55	0	30	1908	May 28	0.41	0	10
1903	July 9	1.06	0	50	1908	May 28	0.49	0	20
1903	Aug. 4	0.84	0	40	1908	Aug. 12	1.98	1	40
1904	April23	0.76	0	50	1908	Aug. 12	0.58	0	25
1904	Sept.18	0.61	0	35	1909	June 22	0.75	0	15
1904	Oct. 5	0.67	0	30	1909	July 2	0.75	0	20
1905	May 11	1.20	0	35	1909	Aug. 14	1.33	0	35
1905	June 4	0.70	0	20	1909	Aug. 14	1.64	1	20
1905	June 25	0.68	0	20	1909	Aug. 27	0.99	0	40
1905	July 28	0.50	0	15	1910	July 22	0.36	0	15
1905	July 28	6.47	0	25	1910	Aug. 23	1.60	0	45
1905	July 28	0.53	0	25	1910	Sept.12	0.45	0	20

Maximum, Aug. 14, 1909, 2.97 inches in 24 hours.

Equations giving the relation of the intensity of rainfall to the length of time of duration have been suggested by engineers for various parts of the country. Curves may be plotted from these equations and compared directly with the records of excessive rains. In general, these equations take the form $i = \frac{c}{t + a}$ in which i = intensity of rainfall in inches per hour, c = constant, t = time of rainfall in minutes and a = constant. Some of the better known formulas are given below:

Locality	Author	Formula	
Boston	Kuichling	$i = \frac{120}{t + 20}$	
East of Rocky Mountain	Talbot	$i = \frac{360}{t + 30}$	Extreme storms
		$i = \frac{105}{t + 15}$	Heavy storms
Baltimore	C. W. Hendrick	$i = \frac{300}{t + 25}$	Extreme storms
		$i = \frac{105}{t + 10}$	Heavy storms
Savanah, Ga.	J. de Bruyn-Kops	$i = \frac{191}{t + 19}$	Extreme storms
		$i = \frac{141}{t + 27}$	Storms occuring once each year.
St. Louis	W. W. Horner	$i = \frac{56}{t + 5^{0.85}}$	
Chicago	C. D. Hill	$i = \frac{120}{t + 15}$	

Probably the most valuable and reliable curve which could be plotted would not depend upon an equation for its form, but upon points plotted from rainfall records for a long term of years.

Some of the heaviest rains recorded have had a largely varying rate of down pour, and from past observation of heavy storms, the rate of rainfall is seldom constant. Obviously, a rain in which the greater part of the rainfall takes place in a few minutes will not present conditions for maximum run-off, unless the drainage area be very small, because the heavy rainfall from different parts of the area will reach the point of concentration at different times. Therefore the rain which will present maximum run-off conditions will be a uniform rain, the duration of which will equal the length of time it takes the water from the most distant point of the drainage area to flow to the point of concentration.

The quantity of run-off of any drainage area is a function of the rate of rainfall, the area, the slope, and general character of the drainage district. The size of the drainage area affects the quantity, the total quantity varying directly as some function of the area, an increase in area resulting in an increase in quantity; and the quantity per unit of area varies inversely as some function of the size of drainage area, because as the area increases, the maximum time of storm water concentration increases, hence the intensity of rainfall decreases. The quantity of storm water varies as some function of the slope of the ground to the point of concentration; the slope of the area affects the quantity in as much as it affects the time of concentration.

In considering the effect of the character of ground upon the quantity, it is well to call to mind the following facts. Before there is any run-off from a drainage area, the soil and vegetation must be covered with a film of water, all depressions and holes must be filled, and the initial absorption of the soil taken up. The initial absorption of porous, sandy, soil will be great, while the initial absorption of a dense clay soil will be small. Ground which is used for agricultural purposes, and is broken up or ploughed will retain a much greater proportion of rainfall than ground which is packed and kept in smooth condition.

The percentage of impervious area is of prime importance in determining the run-off. By impervious area, is meant roofs of houses, sidewalks, pavements. etc. It is necessary to not only calculate the percentage of impervious area at the time of design, but also to make some estimate of and allowance for future improvements, and consequent increase of impervious area.

The foregoing facts relative to the quantity of run-off have been stated as a formula by various engineers for use in different localities. Mr. C. E. Gregory in a valuable paper on "Rainfall and Run-off" page 458, Vol. 58, Trans. Am. Soc. C. E., reduces the more important of these formulas to a common form, in which they may be compared. This tabulation is as follows:

Hawksley .... $Q = A\,C\,R\sqrt{\dfrac{S}{A}}$, in which $C = 0.7$, $R = 1$ to 2

Burkli-Ziegler .... $Q = A\,C\,R\sqrt[4]{\dfrac{S}{A}}$ in which $C = 0.7$ to 0.9, $R = 1$ to 3

Mc Math .... $Q = A\,C\,R\sqrt[5]{\dfrac{S}{A}}$ in which $C = 0.1$ to 0.8, $R = 1$ to 2.75

Hering .... $Q = A\,C\,R\sqrt[6]{\dfrac{S^{1.62}}{A}}$ in which $CR = 1.62$ to 1.05

Parmley .... $Q = A\,C\,R\sqrt[6]{\dfrac{S^{2/3}}{A}}$ in which $C = 0$ to 1, $R = 4$

Adams .... $Q = A\,C\,R\sqrt[6]{\dfrac{S^{1/2}}{AR}}$ in which $C = 1.837$, $R = 1$

Gregory .... $Q = 2.8\,A\,\dfrac{S^{0.186}}{A^{0.14}}$ for impervious surfaces.

Gregory .... $Q = C\,\dfrac{105}{1}$ in which $C = 0.10$ to 0.54

$$\overline{84\sqrt[2]{A\,S^{2}}+25},$$

in which Q = discharge in cubic feet per second

A = drainage area in acres

S = slope of surface in feet per thousand

R = rainfall coefficient

C = run-off coefficient

Hawksley's formula was derived about 1855 for use in London. It first appeared in Roe's Tables and in its original form,

$\log d = \frac{3 \log A + \log N + 6.8}{10}$, expressed the relation between the diameter and slope of a circular sewer, and the size of the drainage area, d being the diameter of a circular sewer necessary to carry off storm water due to a rainfall of 1 inch per hour, and N being length in feet in which the sewer falls 1 foot.

The Burkli-Ziegler formula appeared in 1880, and was intended for use in the design of sewers in German municipalities.

The Adams formula was written in 1880 by Col. J. W. Adams in his book on "Sewers and Drains for Populous Districts" and appeared, as follows: $D^5 = \left(\frac{Q}{39.27}\right)^2 \frac{1}{s}$ in which D = the diameter of a circular sewer in feet, and s = sine of slope of sewer. The Hering formula was prepared in the form of diagrams for use in New York City by Rudolph Hering. Mr. Hering gives the formula deducted from the New York diagrams, as $Q = Ci\ A^{0.833}\ s^{0.27}$.

The Mc Math formula was derived by the author, R. E. Mc Math of St. Louis in 1887. He obtained the formula by making observations of the flow of different sewers in St. Louis and fitting a curve to the observed run-off plotted. The Parmley formula, $Q = Ci\ \sqrt[4]{SA^{5/6}}$ was derived by W. C. Parmley in 1907 in connection with the design of the Walworth Avenue sewer in Cleveland. The Gregory formulae were derived by C. E. Gregory by substitution of values of R and i in the rational formula, Q = Ai R.

The formulae mentioned above are all necessarily empirical, and call for the use of good judgment in the selection of the constants. This use is very dangerous, when attempted by one who adopts "rule of thumb" methods in design. Even when best judgment is exercised the different formulae give widely varying results as to quantity of run-off. Of late years, there has been a decided tendency, on the part of engineers who have had to do with problems of run-off, to make use of the rational formula, or rational method of computing run-off.

The rational formula is Q = A i R in which A = the drainage area in acres, i = the coefficient of intensity of rainfall, and R = the run-off coefficient. The run-off from any area is obviously the product of coefficient of run-off, the intensity of the rainfall, and the drainage area. This method has its value in the fact that R and i must be determined for every drainage area. In this formula, i may be determined from the length of time of concentration or the time it takes for the water to flow from the most distant point of the area to the point of observation, and from a rainfall intensity curve discussed above.

The proper determination of the length of time of concentration is of importance. An arbitrary length of time varying from five (5) to twenty five (25) minutes is usually added to t, to allow for the length of time which is required for the surfaces to become saturated, and for the water to commence to run-off. We speak of impervious surfaces, but as a matter of fact no surface is absolutely impervious. A smooth pavement or a roof when dry, will sometimes require ten minutes to become saturated to the point where the water runs off. A greater length of time is required when the surfaces are pervious.

The length of time of concentration should be determined in each drainage area. To do this, it will be necessary to trace the path of the storm water, and to estimate the distance. The velocity of flow is dependent upon the slope of the ground, and also upon the character of ground, and given these two factors, a reasonable degree of accuracy in computing the velocity should be obtained. The length of time of concentration is the total distance of the path of storm water, divided by the velocity, to which should be added to the initial time allowed for saturation mentioned above in determining i from the rainfall curve.

The factor R is affected by the permeability of the surface, the relation of pervious surface to impervious surface, the length of time of concentration and to a small degree by surface evaporation. The most accurate way of determining R is to measure the run-off for a given area and study it in connection with rainfall curves. In case this is impossible, a study of the soil and other surface conditions should be made, and the value of R estimated, by study of values of R actually obtained under similar conditions. The value of R for impervious surfaces is a function of the time of concentration, because, as has been mentioned above, there is no run-off at the beginning of a rain, and it is a matter of every day observation in the case of a heavy rain on surfaces with a small slope, that the rate of run-off does not equal the rate of rainfall for some little time. Mr. C. E. Gregory in his paper on "Rainfall and Run-off" takes this point up and draws curves showing the values of R obtained for impervious surfaces. These curves were drawn for data obtained by gaugings in eastern cities. Mr. Gregory after analysis of these curves offers the formula $R = 0.175\ t^{\frac{1}{3}}$ for a storm of duration equal to the time of concentration, or $R = \frac{0.175}{t}\left\{T^{\frac{4}{3}} - (T-t)^{\frac{4}{3}}\right\}$ if for greater duration, where T = the time of duration of the storm, and t = the time of concentration. The value of R for pervious surfaces is also a function of the duration of the storm, or the time of concentration. The value of R for pervious surfaces varies greatly with different conditions. During the dry months of the year, when the soil is dry, it will take up a large amount of rainfall. On the other hand, when the soil is saturated or frozen, the pervious surfaces will become practically impervious, and to all intents may be considered impervious.

In Bryant & Kuichling's Report on the Adequacy of the Present Sewerage System of the Back Bay District of Boston, 1909, the following values of run-off coefficient for different classes of surface are suggested:

For water-tight roof surfaces	0.70 to 0.95
For asphalt pavements in good order	0.85 to 0.90
For stone, brick and wooden block pavements with tightly cemented joints	0.75 to 0.85
For same with open or uncemented joints	0.50 to 0.70
For inferior block pavements with open joints	0.40 to 0.50
For macadamized roadways	0.25 to 0.60
For gravel roadways and walks	0.15 to 0.30
For unpaved surfaces, railroad yards and vacant lots	0.10 to 0.30
For parks, gardens, lawns and meadows, depending on surface slope and character of subsoil	0.05 to 0.25

In the problem of determining the proportion of impervious surface to pervious surface a proper allowance should be made for increase in impervious surface which will be the result of increase in population. This increase should be figured to the end of the economic period or period for which the sewer is designed to serve the needs of the community.

Consideration should be given to a comparison of the cost of constructing sewers of sufficient capacity to provide immediate run-off at times of extreme storms, and the cost of the damage or inconvenience occasioned by having the rainfall held on the surface for a short period of time.

With proper consideration of the foregoing points, it is thought that much more accurate results are obtainable through use of the rational formula or method than through the use of other formulas. It may be said that the formulas mentioned are a profitable study in connection with the estimate of R and i.

The determination of the quantity of sanitary sewage to be designed for may be made with a fair degree of accuracy. First it is necessary to estimate the population at the end of the economic period. It is not possible to determine precisely the population of any municipality at a future given date. The past growth of a municipality in question should be studied as it will probably give some idea as to the future growth in population. A study of the growth of different American municipalities will be of advantage. The population increase curve of the average American community will show that, as a city becomes larger the percentage increase in population becomes smaller. In this study it will be probably be profitable to plot a curve showing the increase of population in the past, and as a guide to the probable increase of population in the future, to plot curves of larger cities.

It is well to consider the fact that in addition to the density of population there will be also an increase in the area of a municipality. The general character of a community has a great deal to do with the increase in population. A manufacturing community in general, showing the greatest rate of growth. However, there are instances in communities which have had for their principle industry mines and which have actually decreased in population.

Having determined the population to be designed for, the next step is to determine the probable amount of sewage per capita. In this study knowledge of the water consumption of a municipality is of first importance. Sewage has been defined as the water supply of a community carrying off the human wastes. If this were literally true, a record of the consumption of water in any municipality would give the amount of sanitary sewage directly. Such is not the case, however. There is a large amount of the water pumped into the mains varying between 15% and 50% of the total which runs off into the ground as leakage. A large proportion of water which is used to sprinkle the streets, lawns, etc. never reaches the sewers because being used at dry times of the year, it is absorbed by the surfaces of the pavement or ground. As an index to the amount of water not reaching the sewers, the following table from "American Sewerage Practice" Volume 1 by Metcalf and Eddy is given:

Estimated Quantity of Water Supplied and Not Reaching the Sewers in Milwaukee, 1911.
(Gallons per capita daily)

Steam railroads	5
Manufacturing and Mechanical purposes	5
Street sprinkling	5
Lawn sprinkling	$2\frac{1}{2}$
Consumers not connected with sewers	$7\frac{1}{2}$
Leakage from mains and services	15
Total,	40

The total water consumption was 105 gallons per capita daily.

If a record of the water consumption in the municipality in question is not available, a study should be made of similar communities and an estimate formed as to the amount of water used. From the records of American municipalities, the water consumption, generally speaking, has increased from year to year. While the adoption of meters in a city has generally caused a decrease in the amount of water per capita used, this is usually overcome in the course of a few years. This unit increase is due to the fact that there is an increasing percentage in the number of water connections from year to year. It is well to take this into consideration in determining the average per capita water consumption.

While it is of importance to know the average quantity of water consumption we are more closely concerned with the fluctuations in water consumption as it is the maximum sanitary sewage in which we are interested. There is a fluctuation in the average monthly water consumption, the maximum usually being obtained in the summer months. There is considerable variation in the consumption of water during the day, the typical water consumption curve showing a variation from almost 0 at midnight to 150% of the average at noon. This hourly fluctuation of water consumption has a decided effect upon the rate of sewage flow.

As has been stated before, a study of the water consumption curves daily, monthly and yearly is of great profit in designing for the conditions of maximum sewage flow. There are records in curves available showing water consumption in many different American municipalities which are more properly contained in books and papers on water supply. In general, it may be said that one and one-half times the average daily water consumption may be taken as a fair estimate of the amount of sewage at time of maximum flow. This is only for average conditions, however, and should not be taken as a basis for design unless there is no data at hand.

There is another important source of sewage which is the leakage of ground water into the sewers and appurtenances. This is a matter which has been given but little attention until late years, and in many sewers constructed with no precautions taken to insure water tightness this leakage amounts to a considerable quantity. The elevation of the ground water table above the sewer is an important factor in this leakage, but of still more importance is the method of construction of the sewers. In case of brick or concrete sewers, unless they are made waterproof, the leakage will be great. In tile pipe sewers with cement mortar joints the leakage will be large. Engineers are being converted to the use of water proof joints of various types as will be discussed later on in more detail. There has been little actual data published upon the quantity of leakage in sewers of various types and construction, possibly the most valuable data obtainable is that given in a paper by Mr. John W. Brooks in Transactions American S. C. E., Volume 76, 1909, on the "Infiltration of Ground Water into Sewers," and in the accompanying discussion. This data is summarized in the form of a table which is given below:

Place	Diameter or Dimensions in inches.	Length of Sewers in miles.	Infiltration in Gallons per 24 hours: per foot of joint.	per inch of diam. per mile.	per mile of sewer.
Boston, Mass.	8 to 36	137.00	2.6	1,818	40,000
Mass. State	Various	700.00	....	.....	80,000
Canton, Ohio	....	11.00	....	.....	26,500
Brockton, Mass.	....	16.00	....	.....	25,000
E. Orange, N. J.	....	29.00	....	.....	22,400
E. Orange, N. J.	8 to 24	25.00	0.8	540	8,650
Joint Trunk Sewer, N. J.	....	150.00	....	.....	25,000
N. Brookfield, Mass.	12	0.3	2.0	1,420	17,000
Rogers Park, Ill.	6	1.7	0.3	207	1,240
Altoona, Pa.	27	1.2	2.6	1,510	40,814
Altoona, Pa.	30	0.6	5.0	2,890	86,592
Alliance, Ohio.	15	......	18.7	13,000	195,000
Clinton, Mass.	....	......	....	......	32,500
Concord, Mass.	....	......	....	......	43,000
Framingham, Mass.	....	......	....	......	31,180
Gardner, Mass.	....	......	....	......	45,875
Madison, Wis.	....	......	....	......	48,000
Malden, Mass.	....	38.00	....	......	50,000
Marlboro, Mass	....	......	....	......	59,540
Natick, Mass.	....	....	....	......	88,710
New Orleans, La	....	....	....	......	45,900
Reading, Pa.	....	......	....	......	5,172
Westboro, Mass.	15	0.37	....	88,100	1320,300
Stamford, Conn.	6 to 18	13.38	6.0	4,110	49,340
Stamford, Conn.	6 to 18	13.38	11.3	7,850	94,170
Canton, Ohio	20	......	5.0	3,500	70,000
New Bedford, Mass.	12	......	2.1	1,417	17,000
Fond du Lac, Wis.	24	1.33	1.5	1,010	24,370
E. Orange, N. J.	10 to 24	......	4.7	2,540	43,250
Ocean Grove, N. J.	4 to 12	3.25	2.7	1,890	15,126
Ocean Grove, N. J.	4 to 12	3.25	7.9	5,480	43,764
Newark, N. J.	....	198.7	....	......	115,752

E. Orange, N. J.	24 by 36	......	....	......	570,000
Westboro, Mass.	....	......	....	......	415,850
Altoona, Pa.	33¼ by 44	0.95	....	5,390	264,000
Peoria, Ill.	....	......	....	......	100,000
New York City.	8 to 24	5.00	....	81,400	1300,000
Columbus, Ohio.	42 by 42	0.33	....	120	6,340
°Bronx Valley, N. Y.	44 to 72	9.04	....	123	7,266

° 3 ply felt and pitch 4% water-proofing was used.

In the opinion of the writer it is possible to reduce the leakage to an almost negligable quantity by taking precautions in the construction to making the sewers water-tight. In some cases this effect is not desired, but where it is necessary to pump the sewage water and purify it in sewage treatment plants, it is very desirable to reduce this leakage to a minimum. It is thought by the writer that strict regulation and inspection of the construction of house drains and laterials will reduce this leakage to a considerable extent, as these drains are apt to be poorly constructed, because they are usually laid without proper superintendence.

CHAPTER III.

DESIGN OF COMBINED SANITARY AND STORM SEWER FOR THAT PORTION LYING SOUTH OF 126TH STREET.

(A) Assumptions.

The economic period of design for this sewage system will be taken for about 35 years, because; First, we cannot break with any degree of accuracy the needs of the community beyond the year 1950; Second, a sewerage system which will be adequate for a longer term of years will necessitate too great an expenditure at this time; Third, we have no assurance that the life of a sewerage system will have a greater length than 35 years.

The writer is of the opinion that more accurate results are obtainable by the use of the rational method of determining storm water run-off than by any other method. This is coming into very great use of late years and most engineers who have to do with run-off problems have adopted it. In the design of this combined sewer, the writer will use this method which is expressed by the formula $Q = R\,i\,A$ in which Q = quantity in cubic feet per second, R = the coefficient of run-off, i = intensity of rainfall in inches per hour and A = the area of the district in acres.

In arriving at the intensity of rain-fall designed for, a study of the rain-fall records in Chicago, which has practically the same rain-fall conditions as Whiting, has been made. By referring to the tabulation of excessive rains in Chicago in the last twenty years on page 14, it is found that a very small percentage of the storms recorded will exceed the values given by the formula $i = \dfrac{105}{t + 20}$ in which i = inches of rainfall per hour and t = duration of rainfall in minutes. This formula gives very slightly smaller values than Talbot's formula for intensity of rainfall.

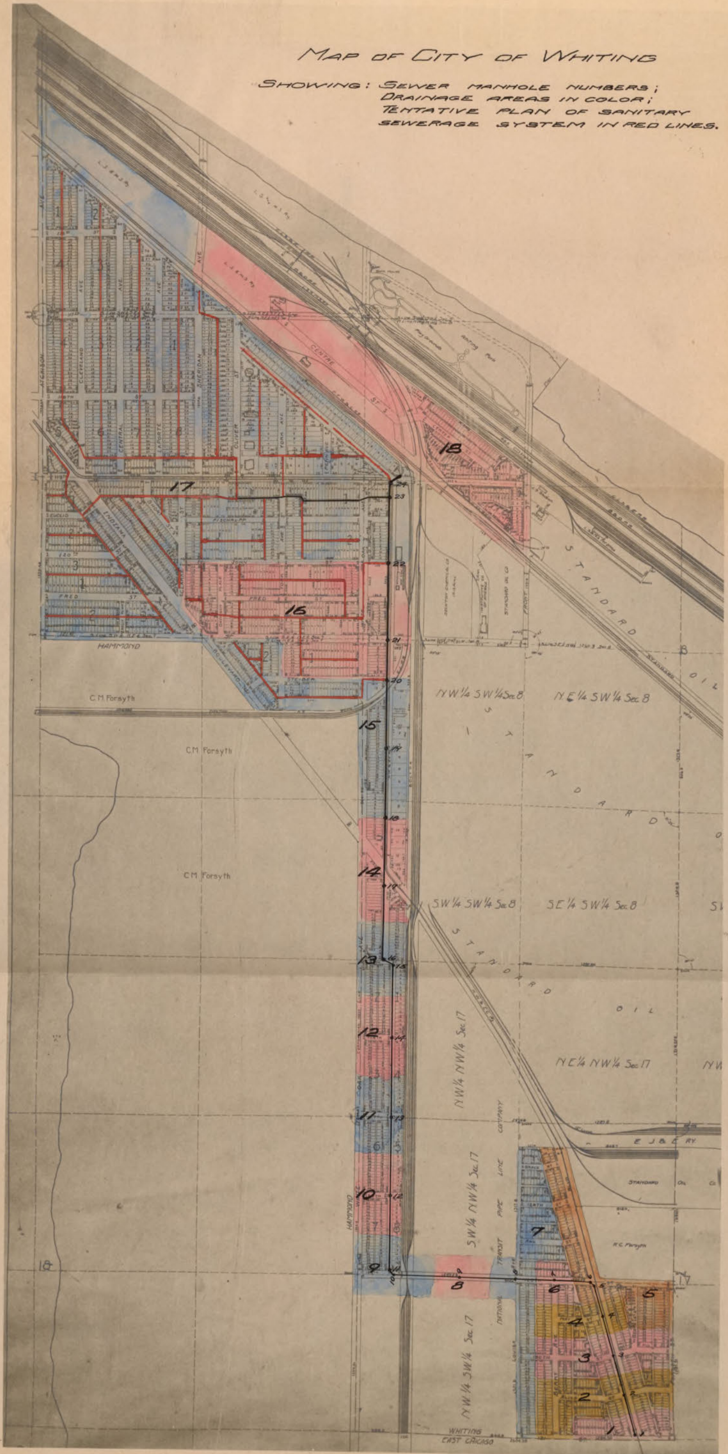
In the judgment of the writer the value of R should be determined by the duration of the rainfall as discussed before. Therefore the formula R = 0.175t will be used. This is the formula suggested by Mr. C. E. Gregory for values of R where the duration of the rainfall does not exceed the time of concentration. The writer recognizes the fact that if the duration of the storm exceeds the time of concentration, the value of R will be increased, but the district to be provided with drainage is of such a nature that the damage resulting from the backing up of the water in extreme storms will be slight. This value of R is chosen and no attempt will be made to use a value of R for cases where the duration of storm is greater than the time of concentration or where the ground is already saturated.

As has been discussed previously, the proportion of impervious to pervious area in any drainage area should be determined with as much accuracy as possible. In determining the impervious area in the district under consideration, the writer has assumed that all of the streets in the district will be paved by 1950 and has assumed that the average width of such streets will be twenty feet. The district is, at the present time, very little developed, the entire population being probably 500 or 600 people and the number of houses about 60. The area of the roofs of the buildings, then, is almost negligable at the present time. From present indications the growth of this district will not be rapid and it is assumed that the impervious area of the buildings constructed before 1950 will amount to about ten (10) per cent of the area of street pavement assumed.

The pervious area will be taken as the area of the district less the area of the street pavements assumed and ten per cent of the area of the street pavements which is the allowance made for roofs. The soil, as has been discussed previously, is of a very porous nature. During the greater part of the year, all of the rainfall will soak directly into the soil as it falls and the rainfall on the sidewalks will be absorbed by the soil before it reaches the pavements. The only condition under which we will have a run-off from this pervious area will be when the ground is frozen during the winter months. At such times the run-off coefficient will still be small due to the flatness of the land.

In determining the time of concentration, it is assumed that there will be an initial amount of time consumed amounting to twenty minutes which will be taken up by the impervious surface becoming saturated and in the time consumed by the water in reaching the sewer. This allowance is rather conservative, as the gutter grades, which have not been established at the present time will probably be very slight. In the determination of the foregoing, the writer has not lost sight of the fact that this run-off will have to be pumped. The cost of installing pumping machinery to take care of the maximum run-off in extreme storms from this district will be greater than the benefit to the district, were immediate drainage afforded in such cases. It is thought that a sewer designed using these assumptions will provide adequate drainage in all except extreme storms.

The Chezy formula for flow of water in conduits has been used in computing the quantity and the Ganguillet and Kutter formula for c is used. From the discussion of the value of n to be used in Ganguillet and Kutter's formula, it would seem that the value of n = 0.013 for vitrified pipe sewer laid with ordinary care


MAP OF CITY OF WHITING
SHOWING: SEWER MANHOLE NUMBERS;
DRAINAGE AREAS IN COLOR;
TENTATIVE PLAN OF SANITARY
SEWERAGE SYSTEM IN RED LINES.
HAMMOND
C.M. Forsyth
NW 1/4 SW 1/4 Sec 8
NE 1/4 SW 1/4 Sec 8
SW 1/4 SW 1/4 Sec 8
SE 1/4 SW 1/4 Sec 8
NW 1/4 NW 1/4 Sec 17
NE 1/4 NW 1/4 Sec 17
SW 1/4 NW 1/4 Sec 17
NW 1/4 SW 1/4 Sec 17
STANDARD OIL
E J & E RY
WHITING
EAST CHICAGO

is good practice. Specifications for construction of this sewer will be rigid and it is probable that a value smaller than 0.013 will be obtained, so that a value of n = 0.013 will be assumed.

(B) Computations.

In this design, the velocity, diameter and slope are taken from the diagram opposite page 10. In the use of the rational method of computing the quantity of run-off a tabulation is most convenient method. In the tabulation given below the manhole numbers and drainge area are as shown on the map opposite. As will be seen on this map, a large portion of the area within the City limits is not taken into consideration. This is due to the fact that this land is owned by the Standard Oil Company and the National Transit Pipe Line Company and will, in all probability, never be sub-divided. It is assumed that there will eventually be sewers along all the streets, which will concentrate at the manholes given for each drainage area. It is also assumed that there will eventually be catch basins at every street intersection which will drain directly into the manholes at that intersection. The existing sewerage system is extended as far south as 126th Street, so that it is necessary to provide only for that portion of the city south of the center of the block between 126th and 127th Street. With the exception of Indiana Boulevard which is paved with water-bound macadam and 129th Street and White Oak Avenue which are paved with asphaltic macadam, none of the streets in this district are paved at the present time and the computed run-off will probably be less than the estimated run-off until the streets are improved. The tabulation is given below and is self explanatory.

Line from	to	Drainage Area	Actual area in acres	Equivalent area in acres	Intensity of rain-inches per hour			AiR in cubic feet per second	Diameter of sewer-inches		Velocity in feet per sec.	Length in feet.	Total time at entrance.	Time of Flow.	Total time.
M.H.	M.H		A	A'	i	R	iR	Q		%S					
1	2	1	0.2	0.2	2.63	0.48	1.26	0.25	12	0.3	1.8	328	20	3	23
2	3	2	0.7	0.9	2.10	0.54	1.14	1.02	12	0.3	2.0	329	30	3	33
3	4	3	1.3	2.2	1.98	0.56	1.11	2.45	15	0.26	2.0	328	33	3	36
4	5	4	0.5	2.7	1.88	0.58	1.09	2.95	15	0.26	2.5	329	36	2	38
6	7	5	1.3	4.0	1.81	0.59	1.07	4.27	21	0.12	2.0	300	38	3	41
7	8	6	0.2	4.2	1.72	0.60	1.04	4.36	21	0.12	2.1	320	41	2	43
8	9	7	0.8	5.0	1.67	0.61	1.02	5.11	21	0.12	2.2	490	43	4	47
9	10	8	0.2	5.2	1.57	0.63	0.99	5.16	21	0.12	2.2	401	47	3	50
11	12	9	0.7	5.9	1.50	0.65	0.97	5.74	24	0.09	1.8	630	50	6	56
12	13	10	0.8	6.7	1.38	0.67	0.93	6.21	24	0.09	2.1	662	56	5	61
13	14	11	0.8	7.5	1.30	0.69	0.90	6.72	24	0.09	2.4	662	61	5	66

(C) Design.

The sewer is designed as shown on sheets 2 and 3 of the plans attached. The grade of the sewer at 130th Street was established at 100.0. This gives a grade of 102.22 at the summit of the sewer at 131st Street, which will provide adequate drainage at that point. It is feasible to build laterals to serve all of this district and provide proper drainage, with this grade established.

The gradients and sizes shown on the tabulation, above, have been used throughout. To provide suitable and proper ventilation, the top line of the sewer is designed to be continuous. This is in accordance with the best practice. At changes in the course of the sewer, approaching 90 degrees, two manholes have been placed so as to minimize the loss of head at the points in question.

As will be noted, the spacing of the manholes is about the maximum of what is considered good practice. In this design, this has been done advisedly to minimize the leakage. That portion of manholes below the ground water plane generally forms a most prolific source of leakage. In order to reduce the leakage, the sewer will be laid with water tight joints, and the manholes will be made water-proof below elevation 100, as will be set forth in detail in the specifications.

No catch basins are provided, as it is expected that they will be constructed when the streets are improved with paving. In the meantime, there is no great necessity for them, and their construction would simply add to the quantity of sewage to be pumped.

CHAPTER IV.

DESIGN OF DEEP SANITARY SEWER IN ALLEY SOUTH OF 119TH STREET AND APPROXIMATE DESIGN FOR THAT PORTION NOW SEWERED.

(A) Assumptions.

There is no reliable data at hand as to the water consumption in the City of Whiting. No guagings have been made of the dry weather flow of the sewerage system. This would undoubtedly be the best basis for design obtainable. With no data at hand on the daily water consumption, or the dry weather flow of sewage, it becomes necessary to arrive at the quantity of sanitary sewage by comparison.

An average of the average daily water consumption of 68 Massachusetts cities and towns given in Metcalf & Eddy's "American Sewerage Practice" is 63 gallons per day per capita. The average maximum daily consumption was fixed to be 198% of the average daily consumption.

In the experience of the writer in suburban towns in the vicinity of Chicago, the average daily water consumption has ranged between 30 and 100 gallons per capita daily. These towns have been metered and the class of inhabitants is fairly good. The population of Whiting is made up largely of foreign born, or children of foreign born people, and it is natural to assume that they, as a whole, will use less water than a more typical American community.

The City has about 50% of the water services metered, and undoubtedly will be entirely metered by the year 1950. This will tend to prevent any radical increase in the quantity of water consumed.

It is estimated by the writer that the present average daily water consumption is 40 gallons per capita, and that this consumption will be increased to 50 gallons per capita in 1950. The maximum daily quantity is taken to be 200 per cent of the average, or 100 gallons per capita. The maximum hourly demand is estimated at 140% of the maximum daily demand or the rate of 140 gallons per capita per day. It will be assumed that 90% of this amount will reach the sewers, so the quantity to be considered in design is 126 gallons per capita per day.

There are 1830 lots in the district under consideration. Assuming the present population to be 6,700, there is an average of about 3.7 people to each lot. In 1950 we assume this district will have a population of 8,700 all connected with the sewage system. As there are 4.76 people on each lot with a maximum quantity of sewage of 126 gallons per capita, per day, there will be a maximum rate of flow of sewage of 600 gallons per day for each lot in the district.

The ground water leakage will not be considered in this design, because in times of maximum flow, if the sewers are flowing full, the ground water leakage will be negligible.

(B) Computations.

With the foregoing assumptions, it is only necessary to have the number of lots in order to compute the quantity. By reference to the map opposite page 30, it will be seen that this district has been divided into areas, in each of which, the sewers will concentrate the sewage and deliver it to the main sewer at a separate point. The tabulation below gives the result of these computations:

Area.	Number of Lots.	Quantity in Gallons per day.	Quantity in Cubic feet per second	Point of Concentration, Manhole Number.
12	60	36,000	0.06	14
13	60	36,000	0.06	15
14	40	24,000	0.04	17
15	135	81,000	0.12	20
16	285	171,000	0.26	22
17	1,150	690,000	1.07	23
18	100	60,000	0.09	The pump pit.

The greater portion of the sanitary sewage from areas 12, 13 and 14 will not be concentrated at any point, but will enter the main sewer along Schrage Avenue by means of house drains. For convenience, the sanitary sewage from these areas has been assumed to concentrate at the manholes given.

(C) Discussion of an Approximate Design for the Entire Area.

A portion of the sanitary sewage in an area 15 will be contributed to the Schrage Avenue sewer along its course. Eventually, however, a lateral sewer will be constructed in Steiber Street which will collect the sewage from the major portion of this area and deliver it to the main sewer at manhole number 20. As will be shown in Chapter 5, the main sewer will have a grade of about 91.0 at manhole 20, and as it is possible to establish the grade of the summit of this lateral at elevation 100 and as the maximum length of the lateral is about 1500 feet, an average gradient of about 0.6% may be used.

Area 16 will probably be served by a main lateral in John Street, with branches in White Oak Avenue, Fred Street and 121st Street. As the lateral will enter the main sewer at manhole 22, at a grade of 90.0, there will be ten feet of head available. The maximum length of any lateral being about 2500 feet, the average gradient possible will therefore be 0.4%.

In area 17, a main sewer will be constructed in the alley next south of 119th Street, which will terminate about on the center line of Oliver Street, extended, as will be discussed later. This point is close to the geographical center of this area. A system of lateral sewers may be designed to enter this sewer as sketched on the map opposite page 30. The maximum length of any lateral, probably the one which will serve the northwest corner of the city, is about 3600 feet. The adoption of a maximum gradient of 0.3% will secure a grade of 103.65 at the extremity of the area to be sewered. While this is not as deep drainage of that extremity as might be desired, it will permit the drainage of a basement of moderate depth.

The sewage of area 18, which as now developed, is a poor and squalid district, may eventually be accomplished by means of a lateral in 119th Street which will empty the sewage into the pump pit directly.

(D) Computations - Alley Sewer.

The maximum quantity of sanitary sewage from area 17, which this sewer will serve, has been determined to be 1.07 cubic feet per second. In determining the size and velocity, the diagram opposite page 10, has been used as before.

It has been the custom of many sanitary engineers to design the smaller sewers as flowing half full. The hydraulic considerations will be unchanged, and the use of a larger size of sewers is permitted. This has been done by the writer in the case of sanitary sewers up to 15 inches in diameter. The arguments in favor of this are, first, that flatter grades may be used to obtain the same velocities, and second, that in places a safety factor of

2 on all assumptions as to quantity, rate of flow, etc. In this design the diagram has been used and the quantity doubled, the quantity used being 2.14 cubic feet per second.

The minimum quantity will probably be the leakage into the sewer. This sewer will receive, at the present time, only the sanitary sewage from the stores and offices abutting upon it. The quantity from this source will be comparatively small, and during the night will be practically nothing. As the sewer will be constructed under the most rigid specifications as to water-tightness, see Chapter 9, the amount of infiltration will also be small. At the upper end of the sewer, there will at no time be sufficient flow to make the sewer self-cleansing.

(E) Design of Alley Sewers.

This sewer has been designed as shown on Sheet 3 of the plans attached. Double strength vitrified pipe will be used due to the depth below the surface of the ground. A 15-inch sewer with a gradient of 0.2% will have a velocity of about 2.25 feet per second, when flowing full or half full. With a smaller quantity flowing, as will be the case when the sewer is first placed in operation, the velocity will probably be a maximum of 2.0 feet per second. There will probably be trouble at this time due to the deposits on the sides of the sewer. It therefore would seem that a flush tank should be provided at the summit. However, as laterals will probably be constructed in the near future to enter at the summit of this sewer, and thereby increase the maximum quantity, it was decided not to provide a flush tank at this point, and to depend upon flushing with a fire hose for cleaning.

A large number of manholes has been provided, which are made necessary by the frequent turns in the alley through which the sewer must run. In reference to the location of the sewer, it may be said that 119th Street is paved with brick on a concrete base and has the existing sewer in the center of the street, water main and gas mains on either side. In addition there is a street car line in the center. The alley is unimproved by paving and there is no underground work to interfere with the construction of a sewer. For these reasons, an alley sewer can be constructed at less cost than a street sewer, and as the connections are shorter it has no disadvantages except that the alignment is poor.

As there are a number of manholes in this sewer, and as they extend well below elevation 100, the permanent ground water plane elevation, it is well to discuss the means taken to secure water-tight manholes. In the opinions of the writer, a manhole constructed of sewer brick masonry, would have to be waterproofed by some means independent of the wall proper, as there is no method known to the writer of constructing a brick masonry wall so as to make it waterproof. Brick as a material was therefore rejected in favor of concrete.

It is well known that concrete aggregate may be proportioned so as to be dense and practically waterproof. To obtain waterproof concrete, extreme care must be observed in proportioning the aggregate to obtain a mixture in which all of the voids in the fine aggregate will be filled by the cement, and all the voids in the coarse aggregate will be filled by the mortar. This mixture is possible in the laboratory, but under actual working conditions, there is a limit which we approach, as to the amount of testing, proportioning and inspecting,which is practicable to secure this ideal mixture. In the handling of the material, particularly where it is deposited in a deep manhole, there is very apt to be a separation of the aggregate which causes voids in the concrete. If for any reason, in the concreting of a manhole, a joint is left from one day to the next, it is a source of trouble and a probable source of leakage. For these reasons, it was decided to use a rich mixture of concrete and secure as dense as possible a mass, but not to rely upon this alone for waterproofing.

The method adopted is as follows: matched lumber will be specified for the outside forms of the manholes. These outside forms will be painted on the inside with a coat of asphaltum. When the manholes are concreted, the outside forms will be left in place. As will be evident, the pressure will always be exerted on the outside of the manholes, which will force the asphaltum into the concrete and effectively secure it from leaking. The additional cost is not great, and is fully warranted in view of the results to be obtained.

CHAPTER V.

DESIGN OF SEWER FROM 126TH STREET TO PUMPING STATION.

(A) Computations.

The computations of quantity of sewerage which will reach this sewer at various points have already been made in Chapters 3 and 4. For convenience the results will be tabulated below:

Area	Line from M.H.	Line to M.H.	% S	Length in feet.	Quantity this area Q	Total Quantity Q'	Diam. of Sewer, inches	Vel. feet per sec.
12	14	15	0.09	630	0.06	6.78	24	2.2
13	15	17	0.09	640	0.06	6.84	24	2.2
14	17	20	0.09	1660	0.04	6.88	24	2.2
15	20	22	0.09	950	0.12	7.00	24	2.2
16	22	23	0.10	550	0.26	7.26	24	2.3
17	23	Pump	0.11	230	1.07	8.33	24	2.4

In the design of a system of sewers, the storage capacity of the sewers is properly considered in the design. Mr. C. E. Grunsky has gone into this problem in detail in a paper on "The Sewer System of San Francisco" in Transactions of the American Society of Civil Engineers, Volume 65, page 294. In this paper Mr. Grunsky works out a rational method of design, in which he takes into consideration the storage capacity of the sewers.

In the design of this sewer, the writer has judged it better practice to let the storage capacity serve as a factor of safety upon the design, rather than to attempt to compute the actual reduction in quantity resulting from the storage capacity. It is of interest, however, to note that the storage capacity of the Schrage Avenue sewer from manhole 11 to the Pump House is 21,000 cubic feet, and the computed quantity at manhole 11 of 6.72 cubic feet per second would flow for 52 minutes before filling the sewer, if there were no additions of sewage further north.

(B) Design.

This sewer as designed, is shown on Sheets 1 and 2 of the plans attached. As will be seen it is to be laid at some depth below the surface of the ground, and also below elevation 100, which is lake level. In this connection, it is well to say a word about the pressure on a pipe in a trench, and the strength of vitrified clay pipes.

Probably the most elaborate and reliable experiments on this subject in this country were made by Prof. A. Marston, and A. C. Anderson. The results of their tests and observations were published in Bulletin 31 of the Engineering Experiment Station of the Iowa State College. These tests were made under a wide variety of conditions, as to nature of soil, width and depth of trench. As a result of their experiments, Marston and Anderson deduced the formula $W = CwB^2$ in which W is the total weight per unit length of pipe, C is a coefficient depending for its value on the ratio of the depth to the width and the coefficient of friction of the backfill against the wides of the ditch, w is the weight of a unit volume of the backfill, and B is the breadth of the ditch a little below the top of the pipe. A tabulation of the approximate maximum loads in pipes in trenches of different width and depth is given below:

Depth of fill above pipe	Breadth of ditch at top of pipe. 1 foot	2 feet	3 feet	4 feet	5 feet
	Dry sand; 100 lb. per cubic foot.				
2 ft.	150	340	550	740	930
4 ft.	220	590	970	1,360	1,750
6 ft.	260	760	1,320	1,890	2,480
8 ft.	280	890	1,590	2,350	3,100
10 ft.	290	980	1,820	2,720	3,650

12 ft.	300	1,040	2,000	3,050	4,150
14 ft.	300	1,090	2,140	3,320	4,580
16 ft.	300	1,130	2,260	3,550	4,950
18 ft.	300	1,150	2,350	3,740	5,280
20 ft.	300	1,170	2,420	3,920	5,550
22 ft.	300	1,180	2,480	4,060	5,800
24 ft.	300	1,190	2,540	4,180	6,030
26 ft.	300	1,200	2,570	4,290	6,210
28 ft.	300	1,200	2,600	4,370	6,390
30 ft.	300	1,200	2,630	4,450	6,530

Depth of fill above pipe	Breadth of ditch at top of pipe. 1 foot	2 feet	3 feet	4 feet	5 feet
	Saturated sand; 120 lb. per cubic foot.				
2 ft.	180	410	650	890	1,110
4 ft.	270	710	1,170	1,640	2,100
6 ft.	310	910	1,590	2,270	2,970
8 ft.	340	1,070	1,910	2,820	3,720
10 ft.	350	1,180	2,180	3,260	4,380
12 ft.	360	1,250	2,400	3,650	4,980
14 ft.	360	1,310	2,570	3,990	5,490
16 ft.	360	1,350	2,710	4,260	5,940
18 ft.	360	1,380	2,820	4,490	6,330
20 ft.	360	1,400	2,910	4,700	6,660
22 ft.	360	1,420	2,980	4,880	6,960
24 ft.	360	1,430	3,050	5,010	7,230
26 ft.	360	1,440	3,090	5,150	7,460
28 ft.	360	1,440	3,120	5,240	7,670
30 ft.	360	1,440	3,150	5,340	7,830

Marston and Anderson also made tests to determine the breaking loads in vitrified clay pipe of various diameters, a summary of which is tabulated below:

Size, inches	Thickness, inches	Breaking load, lb. per lin. ft. Maximum	Average	Minimum
6	0.62-0.75	2,690	1,960	1,690
8	0.70-0.80	3,320	1,940	1,460
9	0.70-0.80	1,970	1,710	1,430
10	0.80-0.88	2,840	1,850	1,210
12	0.85-1.10	3,400	2,120	1,370
15	1.00-1.30	3,890	2,120	1,220
18	1.20-1.50	4,370	2,770	1,570
20	1.3-1.8	4,920	2,910	1,720
21	1.5-2.0	5,600	4,620	3,030
22	1.7-1.7	6,050	5,010	4,530
24	1.3-2.1	5,620	3,360	2,050
27	2.0-2.4	5,940	4,260	3,080

30	2.2-2.7	6,930	5,050	3,530
33	2.5-3.0	6,310	4,620	3,970
36	2.5-3.0	6,340	4,980	3,900

In the application of this data to the problem in hand, an allowance should be made for the fact that the sand foundation and backfill form a splendid bed and insure uniformity of pressure. There will be a maximum hydrostatic pressure of about ten feet on the sewer, which will be properly allowed for by assuming that the backfill is saturated. It is assumed that the width of the trench a little below the top of the pipe will be three feet, which is an allowance of four inches on either side of the pipe. The maximum depth of the trench above the top of the pipe is nineteen feet. With these assumptions, it is seen by reference to the above table, that the maximum load will be 2,865 pounds per lineal foot of pipe. From the above table of breaking loads it is seen that the maximum breaking load per lineal foot on 24 inch pipe was 5,620 pounds, which was double strength pipe; the average breaking load was 3,360 pounds, and the minimum 2,050 which was single strength pipe. It may be noted in the table that the minimum and average values are lower than obtained for any other size between 21 inches and 36 inches. As the maximum load is well below the average breaking load, and as the pipe is to be laid under favorable conditions of loading, it is judged that double strength vitrified clay pipe will sustain the backfill without failure.

As there will be a continual hydrostatic pressure on this pipe, it is fair to expect that the external pressure will always exceed the internal pressure. The joints will be filled with either asphaltum or sulphur as will be more fully outlined in the specifications. The external pressure will tend to force in this type of filler and hold it in place, which is to be desired. In addition, in case there is any settlement of the sewer after backfilling, a plastic filler will conform to any slightly changed position of the pipes.

Provision for future connections is to be made in the manholes at the necessary points. Wye branches for future house connections are also provided at the necessary points.

CHAPTER VI.

PUMPING STATION.

(A) Location.

In the discussion of the selection of the location of the pumping plant, it is well to consider the problem before us as a whole. The problem is to design a storm water and sanitary sewer for the south district of the city, a sanitary sewer for the business district which will eventually become an integral part of a sanitary sewer system as the entire north district of the city.

There are several different plans possible of securing the foregoing results, which will be discussed briefly. A study of the situation makes evident that the most feasible plan for a system of sanitary sewers in that portion of the city now sewered, is one in which all the laterals run to one point to be located at, or near the geographical center of the district in question. From such a point the sewage can be pumped through a force main on to a high level sewer to a place of disposal, or it can be carried through a deep sewer and pumped in conjunction with the sewage from the district south of 126th Street.

The sanitary and storm sewage from this latter district can be carried by gravity to the lake level canal, one mile south of the city limits, but such a plan is not feasible because it would provide but poorly for the needs of the district.

The plans considered were as follows: First; A pumping station in the center of the north district, running the sewage from there either through a force main, or gravity sewer to a point about on the corner of 129th Street and Schrage Avenue. The collection of the sewage of the south district at this point. The construction of a second pumping station at this point, and a gravity sewer to carry the sewage thence to the canal one mile and a quarter south. Second; The collection of the sewage and pumping station in the north district the same as in the first plan, with a force main to carry the sewage direct to the lake. The collection of the sewage of the south district at a point near the north end of the district, the construction there of pumping plant, and a force main from thence to the lake. Third; The collection of the sewage of the north district as in the first plan, a gravity sewer to convey it, thence to a pumping plant on Schrage Avenue. The construction of a gravity sewer to handle the sewage of the south district which would terminate at a pumping plant. The construction of a pumping plant to handle the sewage from both districts, and a force main or gravity sewer to convey it to the lake.

The third plan was selected because it entails the lowest annual cost and having been selected, it is plain that the pumping station should be located at some point north of the intersection of the alley south of 119th Street and Schrage Avenue. The location selected as shown on the map fulfills this requirement and in addition makes a convenient point of entry for a sanitary sewer to provide for area 18 as shown on the map opposite page 30.

(B) Quantities.

The maximum quantity to be pumped has been computed above, page 40, and is 8.33 cubic feet per second plus 0.09 cubic feet per second from area 18, making a total maximum pumpage of 8.42 cubic feet per second. This is of course the maximum quantity to be pumped after the entire sanitary system contemplated is constructed.

The maximum quantity to be pumped immediately after the construction of this sewer is the total of the following: First; The maximum storm water flow for the south district which is (page 31) 6.72 cubic feet per second. Second; The sanitary sewage from the lots abutting on the sewer, which is the sanitary sewer of about 300 lots, having a maximum quantity of sanitary sewage of 0.28 cubic feet per second. Third; Leakage of ground water which, under these maximum conditions is negligible. The maximum quantity to be pumped at present there, will be 7.00 cubic feet per second.

The average quantity to be pumped under ordinary conditions of dry weather flow will be composed of the sanitary sewage from the 525 lots which abut on the sewer, and the leakage of ground water. The average quantity of sanitary sewage per day is assumed to be 300 gallons per day per lot. The total quantity of sanitary sewage will be 157,500 gallons per day. The specifications call for the construction of sewers which will permit of no more than 500 gallons per day per mile of sewer of leakage. As there will be house connections constructed later, which will probably not be constructed with the same standard of excellence, and as tests on existing systems have a much greater amount of leakage than this, it is assumed that there will be a leakage of 5000 gallons per day per mile of sewer, and as there are two and one-half miles of sewer, the total quantity of leakage will be 12500 gallons per day. The average dry weather pumpage will then be 170,000 gallons per day.

(C) Design of Pump Pit and House.

The first consideration in this design is ease and economy of operation, and the second; economy in construction. Automatic control is very desirable, because it effects a considerable economy in operating cost. The only kind of a pumping installation which can be controlled automatically is one with electric motive power. Centrifugal pumps are the most reliable for this kind of service and they lend themselves readily to an automatic control installation, do not have high operating cost, and in addition permit the use of a wet well of small section.

The plans of the pump pit and house are shown on sheets 4 and 5 of the plans. The circular section of the pump pit was chosen because of ease of construction. In the construction of this structure, it is expected that the builder will erect the steel tube on the surface of the ground, on the footing shown. The brick masonry will be built up and the excavation carried on simultaneously, the weight of the masonry being sufficient to sink the structure. When the structure has reached the necessary depth, the six inch concrete foundation will be placed to seal the bottom, the waterproofing asphalt felt carried up over this foundation, and the fourteen inch floor placed as shown. Asphalt felt will be placed between the masonry wall and steel shell as shown and should provide suitable insurance against leakage. The sewer connection will be made by means of a "bull's eye" in the steel shell as described in the specifications.

In the design of a screen, the principle thing to be considered is ease of cleaning. The screen provided, see details on sheet 4 of the plans, can be cleaned by means of long handled rakes worked from the floor of the pump house. The screen is placed in guides as shown, so as to be removable. If any difficulty is experienced in cleaning the screen with rakes, it can be raised and a tray or basket placed in the bottom to be cleaned at necessary intervals. The screen is to be constructed with three-quarter inch openings, which will exclude any matter which might damage the pumps. It is expected that the sewage reaching the well, will contain but very little matter which will be retained on the screen.

(D) Pumps.

The maximum quantity to be pumped when the entire sewerage system is completed is 8.42 cubic feet per second. The maximum quantity to be handled immediately after the construction of this sewer is 7.00 cubic feet per second. The average daily quantity will be 170,000 gallons, which is an average of 0.26 cubic feet per second.

The units to be installed are one six-inch and one twelve inch centrifugal pumps. They are to have a capacity of 800,000 gallons and 3,200,000 gallons per 24 hours respectively, which is at a rate of 1.24 and 4.94 cubic feet per second. The combined nominal capacity of the plant is therefore 6.18 cubic feet per second. The eventual maximum quantity will be 8.33 cubic feet per second, but as this quantity would obtain for only a short time, it is thought this installation will be sufficient.

The six inch pump will be driven by a seven and one-half horse power motor operating at a speed of 900 R. P. M., and the twelve inch pump will be driven by a twenty horse power motor operating at a speed of 600 R. P. M. As the only available power will be alternating current, the motors will be of the squirrel cage type. With the efficiencies to be expected, the power consumption in pumping 1000 gallons against a head of 21 feet will be 1.4 K. W. for the smaller unit and 1.2 K. W. for the larger unit.

The pumps are to be placed on the floor of the wet well as shown on the plans, and driven by motors set on the floor of the pump house. By placing the pumps at the bottom of the well, all necessity of priming is done away with as the pumps will start flooded. A controlling device is provided for each pump which has a float regulator and other appurtenances as detailed in the specifications.

The float for the six inch well is to be set at an elevation about seven feet above the floor of the well, and the float for the larger well at a somewhat higher elevation so that the six inch pump will operate for all average conditions of dry weather flow. The storage capacity of the well will be about 800 cubic feet. This storage capacity was made small purposely to avoid any septic action of the sewage. As the pumps start and stop automatically, there is no disadvantage in the frequent starting of the pump which will be necessary.

A shear valve will be placed in the manhole adjacent to the pump pit, where the sewage can be shut-off in case of break down of both pumps, which is a very remote possibility, or in case it is necessary to remove one of the pumps for repairs. A high water alarm similar to the self starting device is to be provided, which will ring a bell in the fire engine house.

The motors are carried by I-beams, and two sets of I-beams are provided for steady bearings for the shaft, and as hangers for the outlet pipe. The outlet pipes are each provided with a check valve and a gate valve. A special wall casting will be necessary which will be a 6" 12" 18" Wye as shown. The details of the pump pit, pumps, and pump house are given in the plans and specifications.

(E) Cost of Operation.

The total annual quantity of sewage to be pumped can be estimated only roughly. It seems reasonable to assume that a quantity of storm water equal to the entire rainfall on the impervious area in the south district will reach the pumps. The average yearly rainfall in the vicinity of Whiting is 42 inches. This will produce an annual quantity of 8,575,875 gallons.

The average daily quantity of sanitary sewage and leakage as estimated above is 170,000 gallons. This is a yearly quantity of 62,050,000 gallons. Therefore the total yearly pumpage will be approximately 70,000,000 gallons. The centrifugal pumps will consume an average of 1.3 kilowatt hours of electrical energy for 1000 gallons pumped. Assuming the cost of energy to be $0.02 per KWH, the total annual power cost will be $1,820.00.

The pump pit, pumps and pump house are estimated to cost $7,500.00. The yearly depreciation will be taken at five per cent of this first cost, and with an interest rate of five per cent, the total of these two charges will be ten per cent of the cost or $750.00. As the plant will be automatic in operation, only occasional attendance will be necessary to clean the screens. It is thought that a fair allowance for attendance and for incidental repairs is $100.00 per year.

To resume, the total annual cost of pumping will be as follows:

Power	$1,820.00
Interest and Depreciation	750.00
Attendance	100.00
Total annual cost of operation,	$2,670.00

The quantity of sanitary sewage and infiltration when the entire system is completed, using the same assumption as above will be approximately 214,000,000 gallons per year. The storm water sewage will be the same as above, so the total annual pumpage will be

222,000,000 gallons. The energy cost of pumping this quantity is $5,722.00. The total cost of pumping will be as follows:

Power	$5,772.00
Interest and Depreciation	750.00
Attendance	100.00
Total annual cost of operation,	$6,622.00

(F) Outlet.

Because of the danger of having a vitrified pipe sewer too close to the surface under the several railroad tracks to be crossed, a cast iron pipe sewer eighteen inches in diameter will be used for an outlet sewer from the pumping plant to the lake. An additional point in favor of the use of a force main is in case it is necessary to construct and operate sewage purification works at any time, the additional head to operate such work may be obtained with this pumping plant as designed with a few miner changes.

The total head to be pumped against in a plant of this nature is the sum of the friction head in the suction pipes, the actual head the sewage is to be raised, and the friction head in the outlet pipe, less the head due to the elevation of the surface of the sewage is the wet well above the pumps. In this plant, there are no suction pipes, the actual maximum elevation the sewage is to be raised is 18.25 feet. When this maximum elevation head is to be pumped against, the well is almost empty. Since the quantity to be pumped is comparatively small, the friction head in the outlet pipe will also be small. When there is any quantity to be pumped which approaches the maximum, the total head to be pumped against will be reduced by the head due to the elevation of the sewage in the wet well. The specifications have therefore been drawn for pumps to operate at their maximum efficiency when pumping against twenty-one foot head.

The location of the outlet will be close to the existing outlet as shown on the map opposite page 3.

CHAPTER VII.

ESTIMATE OF COST.

(A) Construction Methods.

It is clear, from a knowledge of the water soaked sand in which the sewers are to be laid, that a great deal of difficulty with water will arise during construction. The rate of flow of water through this material is very rapid and if the contractor should attempt to construct the sewers by any ordinary methods, such construction would probably prove very costly. The best method of construction known to the writer is the one described in Engineering and Contracting of August 5, 1908, on the Cost of a 7-Ft. Brick Sewer in Gary, Indiana. Extracts from that article are given below:

"In trenching for a 7-ft. sewer through water soaked sand at Gary, Ind., the sand is being unwatered by driving well points and pumping. The method has enabled what promised to be a difficult task to be accomplished with comparative ease. Only a moderate amount of sheeting has been necessary and practically no caving has resulted.

"The sand through which the work passes is very fine, such a sand as forms the dunes of Michigan and other states bordering Lake Michigan. When water soaked it takes a slope of about 1 on 15. At Gary this fine sand is water soaked to within a few feet of the surface; in places water covers the surface. So far as excavation work goes the material is to all intents and purposes a quicksand.

"In brief, the method of work adopted is as follows: A wide shallow trench is excavated by a drag-scraper bucket-excavator of the Page & Shnable type to about water level, say to a depth of 6 to 8 ft. Bleeding is then begun. A 4-in. pipe 132 ft. long in six 22-ft. sections is stretched along the center line of the sewer. On each side of this pipe about 3 ft. away is sunk a row of well points 2 ft. apart. These well points are 3 ft. long and are attached to 13-ft. pipes. The top of the driven pipes are connected by hose to the 4-in. pipe line which has cross-valves for the purposes. A pump connects with the 4-in. pipe line and also with a 4-in. well point sunk vertically underneath. An extension of the 4-in. pipe line with strainer end also takes the surface water from a sump.

"This battery of well points lowers the water so that a further excavation of 6 to 8 ft. can be made between sheet piling.

A second battery of well points is then sunk at this new level. In this battery, however, the points are sunk close to the sheeting and each row feeds into a separate 2-in. pipe along the trench. This battery lowers the water level enough to permit excavation to sub-grade, which is some 6-ft. below the bottom of the sheeting. The brick sewer is then built in the usual manner and the backfilling done by means of a derrick and Hayward clam shell bucket .

"The first battery of well points occupies a narrow space along the center of the trench; this permits the sheeting to be driven outside of the well points. The well points are 2 ins.x 3 ft., and they are attached to 2-in. x 13-ft. pipes with ells at their tops. A 4-ft. length of wire lined hose is attached to each ell. These points are sunk vertically by jetting. Two men were timed in jetting. They used 1-in. jetting pipes with about 100 lbs. water pressure and sunk four points in one minute. This time did not include making connections. In addition to the two rows of 2-in. points, a 4-in. point is sunk directly under the pump.

"The well points are connected by the short hose lengths to a 4-in. horizontal suction pipe. Six 22-ft. sections of suction pipe are used with flanged joints. Each section has 11 cross-valves with double bushings for the hose connections. A gate valve near the end of each section permits the rear-sections to be removed and placed ahead as fast as the work progresses. An extension of the 4-in. suction pipe forward to a sump in the excavation being made by the scraper bucket handles the surface water.

"The water is drawn from the suction pipe by an Emerson No. 3 pump with 5-in. suction and 4-in. discharge. The pump is hung to a chain fall from an A-frame mounted on rollers. It discharges into a tile drain alongside the trench; this drain leads back to the completed sewer discharging behind a temporary dam of bags of sand inside the sewer. Summarized, the first battery of well points is composed as follows:

1 No. 3 Emerson pump.
1 4-in. well point sunk below pump.
132 3-in. well points sunk in two rows.
1 4-in. suction pipe with extension to surface water sump.

" The trench is sheeted 10 ft. wide, the sheeting being carried along so as to embrace about one section (the rearmost) of the first battery of well points. The sheeting is 2 x 8-in. x 12-ft. planks and is driven by mauls. Waling pieces and trench braces are placed as the excavation proceeds. This excavation is carried down about 6 ft. by shovelers and at this level the second battery of well points is placed. The sheeting is pulled as the backfilling proceeds.

" The second battery of well points consist of two rows like the first, but the rows are placed wide apart (close inside the sheeting on both sides) and each has a separate suction pipe. The suction pipes are 2 ins. in diameter and the well points are $1\frac{1}{4}$ ins. in diameter; the well points and pipes are 16 ft. long and when sunk they penetrate a couple feet or so below sub-grade and

6 ft. below the bottom of the sheeting. The suction points are made in sections with hose connections every two feet and gate valves at the ends.

"Two pumps operate the second battery of well points; they are of the same size and make as that for the first battery and are suspended similarly. Each pump draws water from both rows of well points and also from a 4-in. well point sunk directly under the pump. This is accomplished by means of a four-way connection in the suction of each pump, about 1 ft. below the pump. From this connection 2-in. pipes branch right and left to connections with the 2-in. suction pipes and a third connection is made with the 4-in. well point. Operating in parallel the two pumps can, by means of the gate valves, concentrate their work on those portions of the battery of the well points where especially large quantities of water are encountered or can pump from the whole system, also either one of the pumps can be cut out. These pumps discharge into the same tile drain as the first pump.

"The methods of advancing the second battery of well points is substantially the same as for the first; that is, the rear sections of suction pipe and well points are detached and placed in front. Generally the forward end of the second battery is kept far enough ahead to overlap the rear section of the first battery.

"The deepening of the trench at the rear end of the second battery of well points is done by hand. So perfect is the drainage that it is found possible to excavate some 6 ft. deeper than the bottom of the sheeting, and to construct the brick sewer in the trench bottom with no more seepage than can be handled by a fourth Emerson pump, which takes water from a sump and discharges behind the temporary sand bag dam mentioned previously."

The sewer described above is much larger than this Whiting sewer, and undoubtedly the details of the pumping and excavating plant would have to be altered somewhat to fit the smaller trench in the construction. The writer has knowledge of the same bleeding method being used in the construction of all sizes of sewers at Indiana Harbor, where the same general conditions exist as at Whiting and Gary. It may be said that the topography of these three localities is practically identical.

(B) Cost Data.

The cost data on the construction of these sewers will be collected under the following heads: Excavation and backfilling, sheeting, pumping, laying and materials. Lack of space prevents the presentation of data on the minor elements of cost in this construction.

In the application of any available cost data to this estimate, a method of construction similar to that described above will be figured on. The trench can be excavated down to a point a little below the water line by some sort of scraper-bucket machine. The cost of scraper excavator work given in the above article was $0.55 per cubic yard. The cost of drag-bucket excavator work given in Engineering and Contracting of January 27, 1909 was $0.061 per cubic yard. This was also work in Gary, Indiana, and in the construction of a wide trench. The cost of sewer excavation by means of a derrick was given in Engineering and Contracting, September 8, 1909 as about $0.25 per cubic yard. The work was at Big Rapids, Michigan, and the material excavated was gravel and boulders. The trench was four feet wide and varied between 14 and 18 feet in depth

The trench below the ground water plane will be excavated by hand. In nearly all cases the material will have to be handled at least twice. In Engineering and Contracting of October 7, 1908 the cost of this kind of work at Gary is $0.57 per cubic yard. In Engineering and Contracting of January 27, 1909 the cost of this class of excavation was $0.39 per cubic yard. It was stated that the material was scaffolded an average of four times, and the laborers were paid $2.25 per ten hour day. Those latter conditions closely approach the Whiting conditions.

The backfilling cost on the last mentioned work was $0.07 per cubic yard. It is stated in an article on sewer construction at Atlantic, Iowa, in Engineering and Contracting, May 15, 1907, that the backfilling, which was done by a team and a drag scraper, cost $0.03 per cubic yard. The work was in heavy black soil.

The cost of sheeting at Peoria, Illinois, is given in Gillette's "Handbook of Cost Data" on page 802. Sheeting in 16 foot lengths a gang of 22 men, with daily wages of \$36.00 sheeted 12 lineal feet of trench 45 feet deep per day. This is equivalent to a cost of \$0.06 2/3 per lineal foot of trench for each foot of depth. There were about 192 feet B. M. of sheet plank per lineal foot of trench and probably 38 feet B. M. of stringers and braces. If 2 inch sheet plank were used, the driving, pulling, and bracing cost about \$13.00 per 1000 feet B. M. The cost of the same work using an Adams trench machine was practically two thirds the cost of the hand work, given above. The cost of hand work on a trench 35 feet in depth was \$0.05 per lineal foot per foot of depth. Smaller trenches, 8 to 16 feet deep in sand cost from \$0.01 to \$0.02 per lineal foot per foot of depth for sheeting.

In Engineering and Contracting of August 5, 1908, the cost of sheeting at Gary, Indiana, is given as follows:

"Two rows of 2 x 8-in. x 12-ft. sheeting 60 ft. long are driven, braced and pulled per 9-hour day with the following gang;

4 men setting braces at \$2.25	\$ 9.00
3 men driving sheeting at \$2.50	7.50
4 men pulling sheeting at \$2.50	10.00
1 carpenter at \$3	3.00
Total	\$29.50

This gives a cost of 24½ cents per lineal foot of 12-ft. sheeting driven, braced and pulled, not including materials and superintendence, etc."

The cost of sheeting at Gary, Indiana, in a 17 foot trench was 37 cents per lineal foot of trench for driving, pulling, bracing, and hauling ahead. The work was in sand and was done by hand. The cost per 1000 feet B. M. was practically \$5.00

With hand driving it is generally estimated that sheeting can be used four times and more with machine driving. The cost of sheeting at the present market price is $20.00 per 1000 feet B. M.

The only cost data on pumping by means of well points is given in the three articles on sewers at Gary, Indiana, mentioned above, and is as follows:

" The pumping is continuous day and night, but the jetting of well points and changing of piping is confined to the regular shift of 9 hours. The gang worked is as follows:

14 pipe line men at $2.25	$31.50
10 firemen (two shifts) at $3................	30.00
2 foremen at $3...............................	6.00
6 laborers at $2	12.00
Coal for 24 hours (estimated)	15.00
Total	$94.50

"This gives a cost of $1.57 per lin. ft. of trench, not including superintendence, interest, depreciation, etc."

"The item of pumping comprises all the work of sinking and shifting the well points and pipe line and the removal of the backwater in the finished part of the sewer. Three Emerson pumps took water from the well points, a fourth handled the backwater and a duplex pump furnished water for boilers, mixing mortar, jetting, etc. The cost was as follows:

Item.	Total	Per lin.ft.
Laborers, 542 days, at $1.75	$ 948.50	$0.2227
Pipe line men, 958 days, at $2.50	2,395.00	0.5625
Total for pipe work	$3,343.50	$0.7852
Coal, 100 days, at $15	$1,500.00	$0.3499
Firemen, 855 days, at $3.50	2,992.50	0.7025
Total for pumping	$4,492.50	$1.0524
Grand total	$7,836.00	$1.8376

"Pumping costs and pipe line costs have been separated, since the first is a continuous expense which does not vary from day to day, and the second cost of operative only when construction is actually going on.

"The pumping plant consisted of 3 No. 3 Emerson pumps drawing from the well points; 1 No. 2 Emerson pump taking water from the pools formed behind the drag bucket excavator; 1 duplex pump for boiler feed, jetting points, wetting brick, etc., and 4 30-hp. horizontal boilers mounted on wheels. This plant worked continuously. The cost of operation was as follows:

Item.	Total
Laborers, 464 days, at $2	$ 928.00
Fireman, 439 days, at $3.50	1,536.50
Pipe linemen, 1,238 days, at $2.50	3,094.00
Foreman, 27 days, at $3.50	94.50
Coal, 60 days, at $15 (estimated)	900.00
Total	$6,553.00

"This gives a cost per lineal foot of sewer of $1.61 for pumping. Charged entirely against the excavation between sheeting which was closely 12,893 cubic yards, the cost of pumping per cubic yard of excavation was 50.8 cents."

In the first example the trench was 10 feet wide by 13.5 feet deep. In the second the trench was about 10 feet wide and ranged from 18 to 30 feet in depth. The third trench was about 10 feet in width and averaged 17 feet in depth.

Cost data on laying pipe sewers in St. Louis is given in Gillette's "Handbook of Cost Data", page 861. The work was done by contract during the three years from 1901 to 1904, there being 40 miles of vitrified pipe sewers constructed from 12 to 24 inches in diameter.

"Four men, the bottomman and his helper, with two men handling and lowering the pipe, laid 21-in. and 24-in. pipe at the rate of sixteen lineal feet per hour, at a cost of 6 cents per lineal foot. Three men will lay the same amount of 15-in. or 18-in. pipe in the same time. Including the material for jointing, the cost of laying pipe is 10 cents per lineal foot."

The following data is given in Gillette's "Handbook of Cost Data", page 868, on a short section of sewer.

"Pipe and Pipe Laying (262.5) lin. ft.)

	Total	Per lin.ft.
Foreman, 10 hrs., at $0.50	$ 5.00	$0.02
Labor, 120 hrs., at $0.25	30.00	0.11
Bottomman, 63 hrs., at $0.30	18.90	0.07
Cement, 1-150 bbl., at $1.50	.....	0.01
Pipe, per ft.	.....	1.25
Total		$1.46
Excavation per lin. ft.		0.20
Grand total per lin. ft. pipe sewer		$1.66

The cost of sewer pipe F. O. B. Whiting, at the present market, is as follows:

Double strength	24 inch	$0.82	per lineal foot	
"	"	21 "	0.57	" " "
"	"	18 "	0.38	" " "
"	"	15 "	0.27	" " "
Single	"	12 "	0.20	" " "

The cost of Wye branches or Tees 2 feet long is practically double the cost of straight pipe 2 feet long.

(C) Estimate.

The unit costs used in making up this estimate are as follows:

Scraper excavation, per cubic yard	$ 0.15
Hand excavation, per cubic yard	$ 0.40
Backfilling, per cubic yard	$ 0.03
Driving, pulling, bracing sheeting, per lin. ft. per ft. of depth	$ 0.03
Sheeting at $20.00 per M., per ft. of trench,	$ 0.16
Pumping cost, labor and fuel to elevation 100, per lin. ft.	$ 1.00
For each additional ft. of cut, per lin. ft.	$ 0.05
Pipe laying, labor and joint filler, per inch of diameter, per lin. ft.	$ 0.02
24 inch pipe on the work, per lin. ft.	$ 1.00
21 inch pipe on the work, per lin. ft.	$ 0.85
18 inch pipe on the work, per lin. ft.	$ 0.65
15 inch pipe on the work, per lin. ft.	$ 0.55
12 inch pipe on the work, per lin. ft.	$ 0.50
18 inch cast iron pipe, per lin. ft.	$ 3.80
Labor laying 18 inch cast iron pipe, per lin. ft.	$ 0.05
Lead for joints in 18 inch cast iron pipe 30 pounds per joint at $0.05 per lin. ft.	$ 0.12
Yarn used in laying 18 inch cast iron pipe, per lin. ft.	$ 0.08
Brick manholes 7 ft. deep, each	$30.00
Concrete manholes per ft. of depth below elevation 100,	$ 5.00
Macadam paving, per sq. yd.	$ 1.25
Asphaltic macadam paving, per sq. yd.	$ 2.00

It will be noticed in the above that only the larger and more important items of cost have been worked out in detail. In computing the quantity of excavation, the width of trench will be taken as 2 feet wider than the diameter of the pipe down to elevation 100, and below that 1 foot wider than the diameter of the pipe. The sheeting has been estimated to be used four times.

Estimate.

10,500	cubic yards of scraper excavation, at \$0.15 per cubic yard	\$ 1,575.00
7,500	cubic yards of hand excavation, at \$0.40 per cubic yard	\$ 3,000.00
18,000	cubic yards of backfilling, at \$0.03 per cubic yard	\$ 540.00
11,090	lineal feet of trench sheeting and bracing, average depth 16 feet, at \$0.19 per lineal foot	\$ 2,107.10
10,375	lineal feet of trench drained by pumping, average depth 6.6 feet below elevation 100, at \$1.33 per lineal foot	\$13,798.75
6,714	lineal feet of 24 inch pipe sewer, laid in place including labor and materials at \$1.48 per lineal foot	\$ 9,936.72
1,648	lineal feet of 21 inch pipe sewer, laid in place including labor and materials, at \$1.25 per lineal foot	\$ 2,060.00
2,074	lineal feet of 15 inch pipe sewer, laid in place, including labor and materials at \$0.85 per lineal foot	\$ 1,762.90
654	lineal feet of 12 inch pipe sewer, laid in place including labor and materials at \$0.74 per lineal foot	\$ 483.96
32	manholes, brick above elevation 100, concrete below elevation 100, average depth of 13 ft. at \$60.00 each	\$ 1,920.00
1,300	lineal feet of 18 inch, Class A cast iron pipe, at \$4.05 per lineal foot	\$ 5,265.00

	Pumping Station, pit	$ 2,000.00
	Pumping Station, building	$ 2,000.00
	Pumping Station, pumps and equipment	$ 3,500.00
3,450	square yards of water bound macadam paving at $1.25 per square yard	$ 4,312.50
700	square yards of asphaltic macadam paving at $2.00 per square yard	$ 1,400.00
	Cost,	$55,661.93
	Superintendence, engineering, contingencies and profit 25%	$13,915.07
	Total estimated cost,	$69,577.00

CHAPTER VIII.

CONCLUSION.

It may appear that Chapter 2, on "General Principles of Sewer Design", has been given a more elaborate treatment than is warranted by the length of the thesis. To the writer, this is the most important part of the subject and if anything, should be more elaborate. The theory and facts upon which the solution of any problem rests are fully as important as is the use of good judgment in the working out of the problem. After a careful perusal of all available data on the subject, one is impressed by the meagreness of the data in comparison with the vast amount of work done. Sufficient theory and data have been given to form a basis for the assumptions made in the following chapters.

In the design of the storm water sewers some rather unusual assumptions have been made in regard to run-off, which result in the selection of a relatively small size of sewer. The assumption that there will be no run-off from any of the district classed as pervious will probably be questioned by the Engineer who has no first hand information as to the topography of the land. This sandy soil, which so freely transmits water, forms a large storage reservoir for rainfall. The percentage of voids in the soil, which is made up of sand grains of a fairly uniform size, is probably over 30%. It is seen that the fall of one inch of rain will only raise the ground water table approximately three inches. Therefore the writer is convinced that the assumption made is correct except at times when the ground is frozen, or in early spring when the melting snow and ice has saturated the soil.

It is difficult to estimate the run-off under such conditions as there ~~is~~ no data available as a guide. However, it is safe to say that, if the design were made to handle the run-off at times when the ground is frozen or saturated, a much larger sewer would have been necessary as well as a larger capacity at the pumping plant. As nearly as can be foreseen at this date, there will be no material damage resulting from the design made. In the worst conditions possible, the water will be backed up, and possibly held on the streets for three or four hours, which will entail some inconvenience. The community served is made up of foreign born people, who have built a rather poor class of houses. While this is no reason for providing inadequate drainage, it must be considered for economical reasons. It is not only the duty of an Engineer to make adequate designs, but he must also make the best design possible for the lowest cost. In works of this nature, he must stop short of the point where the additional dollar spent will not produce an additional dollar's worth of benefits.

The sewers as designed will be self-cleansing with the exception of the alley sewer. When the entire sanitary sewer system is constructed, the alley sewer will be self-cleansing. While we are not concerned with the lateral sanitary sewers, at this time, it is probable that the limiting grades, outlined in Chapter 4, will necessitate the use of flush tanks at the summits of the laterals.

In regard to the failure of sewer pipe, the only failure known to the writer in conditions at all similar to these occurred at Gary, Indiana. The sewer was a 24 inch vitrified clay pipe laid in sand. After being laid with a few feet of backfill, the grade of the street, in which the sewer was laid, was raised some ten feet, the filling being done from one side. As a result, the load was unbalanced. The failure was caused by spring floods, which caused an internal pressure of about eight feet. This condition would be impossible in this Whiting sewer, because there is a constant external hydrostatic pressure on the pipe, which is greater than any probable internal pressure.

It will be observed by reference to the tentative sanitary sewer design shown on the map opposite page 30, that the location of the sewers is largely in the alleys. This should result in a considerable reduction in their cost, as all of the alleys are unpaved. In addition there will be no difficulty with existing underground structures to be contended with, except at street crossings.

No attempt has been made to design the pumps, motors, or controlling devices. For a small installation, the cost of making such designs is expensive out of all proportion to any saving which might be effected. It is generally considered good practice to specify only general conditions to be met, and allow the manufactures to supply their own plans. Rigid tests should be made after the pumps are installed to make certain that they operate at the economies guaranteed by the maker.

This installation may be criticized on the ground that there is no reserve pumping unit for use in case of failure of one of the pumps. The maximum conditions will occur only occasionally, and as a matter of fact, we have a reserve unit, except during times of heavy rainfall. However, it may be that an additional unit will be needed in order to insure reliability of service; and if such proves to be the case, the additional unit can be installed in this well, with no difficulty.

The method of construction of the pump pit somewhat briefly outlined in Chapter 6 is perhaps somewhat unusual. The writer had charge of the construction of a small settling tank at Palatine, Illinois, which was constructed in a similar way. The tank was rectangular in section and was made of concrete. The soil was a rather stiff clay mixed with gravel. In that case no steel shell was used. The depth of the tank was about 28 feet, forms being set and concrete poured in sections of 7 feet. The earth was then excavated from inside and the tank allowed to settle of its own weight until the required depth was reached.

No attempt has been made to solve the problem of sewage disposal as the popular opinion in Whiting is that it would be an unnecessary expense at this time. As the conditions are, the refineries of the Standard Oil Company cause far more pollution of the lake, than does the sewage of the City of Whiting. At such a time as the State or Federal authorities compel the purification of all sewage entering the lake, a site for a plant is available on the city's land near the outlet.

The subject of sewer design, even when applied to small works, is so comprehensive that the writer has not attempted to discuss thoroughly any of the various places of the subject. It is hoped, however, that the discussion has been sufficient to make clear the reasons for the designs made.

The portions of the specifications given are those portions which apply to special classes of work or conditions. Those clauses which are common to all sewer specifications have been omitted, as they present no features of special interest.

CHAPTER IX.

PLANS AND ABSTRACT OF SPECIFICATIONS.

EARTH EXCAVATION ITEM 1.

Description.

4. The Contractor shall make to the lines and grades given by the Engineer, all excavations required for constructing the sewerage system, including the laying of sewer pipes, specials, branches, etc., the placing of manholes, pump pit, and other appurtenances, and shall excavate any other material which, in the opinion of the Engineer, is desirable for any purpose appurtenant to the construction of the work.

Clearing.

5. The site of all excavation shall be first cleared of all trees, lumber, walks, stumps, brush and rubbish, which shall be removed or disposed of in a satisfactory manner. All surfacing material from excavations, including sidewalks, curb, pavement, gravel, road metal, sods, loam, or other special material shall be carefully removed and kept separate to be used in resurfacing the trenches as may be directed by the Engineer.

Trimming.

6. Trenches shall be excavated of sufficient width to afford ample room for building and inspecting the structures they are to contain, and for timbering, pumping and draining, and also for the removal of any material not deemed suitable for foundations by the Engineer.

Placing of Sewers.

7. All sewers, whether vitrified or cast iron, must be laid on a good foundation, trimmed to shape, and where required, secured

against settlement in a manner approved by the Engineer. The sewers shall be laid to the lines and grades given by the Engineer, who will thereby indicate the depth of excavation. At bells or special joints enough additional depth shall be excavated to permit the making of the joint in a proper manner.

Unauthorized excavation.

8. Wherever the excavation is carried beyond or below the lines and grades given by the Engineer, the Contractor shall, at his own expense, refill all such excavated space with such material as may be directed, in order to insure stability of various structures. Beneath concrete structures, vitrified and cast iron pipe sewers, the filling of space excavated without authority shall be completed thoroughly by tamping or flooding, or if deemed necessary by the Engineer, shall be refilled with concrete at the Contractor's expense.

Additional excavation.

9. It is expected that satisfactory material for foundations will be found at the elevations shown on the drawings, or specified herein, but in case the materials encountered are not suitable, or in case it is found desirable or necessary to go an additional depth, the excavation shall be carried to such additional depth as the Engineer may direct. Additional depth of excavation so ordered, will be paid for under the respective unit prices herein specified.

Shoring.

10. In trenches and other excavations where sheet piling, sheeting, shoring, or other supports are necessary, they shall be satisfactorily furnished, placed and maintained by the Contractor.

11. Removal of timbering.

If required, all timbering or other supports shall be removed and all voids carefully filled as directed. If any timbering is ordered in writing by the Engineer to remain in place, such timber shall be cut off as directed and that remaining in place will be paid for as an extra at the rate of Fifteen ($15.00) Dollars per thousand feet, board measure, but no allowance will be made for loss or waste.

Removal of water.

12. The Contractor shall at all times during construction, provide a suitable pumping or bleeding plant with which to remove and properly dispose of all water promptly from all excavations and keep them dry until the structures to be built therein are completed, and the concrete or mortar is sufficiently set to allow water to come upon it.

Disposal of material.

13. All material excavated shall be disposed of in backfilling, making fills, grading around the work, as may be directed, to the lines and grades given by the Engineer, or hauled away by the Contractor to a place of disposal of his own choosing as the Engineer may direct.

Backfilling.

14. All trenches or excavations around structures shall be backfilled up to the original surface of the ground or to such grades as shall be directed. The backfilling shall be done as compactly as possible, and the material shall be flooded in such a manner as to prevent after settlement. The filling shall be made with the best material available, selected especially for the purpose and free from stones over three (3) inches in diameter.

35. VITRIFIED PIPE SEWERS. ITEMS 3, 4, 5 & 6.

The Contractor shall furnish and lay, as required, vitrified sewer pipe including "Wye" branches in the sewer trench at such other points as may be required, including all bonds, branches or other specials required; laid with joints filled with tarred oakum and asphalt, special joint-compound or sulphur and sand as required. All sewer pipe shall be of the best commercial grade of standard vitrified pipe, with socket joints, cylindrical in form, smooth, sound and thoroughly burned, well glazed and free from imperfections. All pipes and specials that have injurious fire-cracks, blisters or pimples, or that show transportation, cooling or frost cracks or that betray in any manner a want of thorough vitrification of fusion or the use of improper or insufficient materials or methods in their manufacture, will be rejected.

36. The pipes shall be at least two and one-half (2½) feet in length, except that shorter or cut lengths may be used to make closures and all pipes and specials shall practically conform to the shape and dimensions of the original pattern. All hubs and sockets must be of deep and wide socket dimensions and of sufficient diameter to receive, to their full depth, the spigot end of the next following pipe or special, without any chipping whatsoever of either. Pipes and specials which cannot be thus freely fitted will be rejected.

Laying.

37. All pipes before being laid, or lowered into the trench, shall be fitted together dry and matched so that when laid or jointed in the trench they shall form a smooth and true line. All pipes shall be laid true to the lines and grades given by the Engineer. The ends or shoulders of each pipe shall abut against the next adjacent pipe in such a manner that there will be no unevenness of any kind along the bottom of the pipes.

Joints.

38. The joints shall be fitted and made water-tight first by the use of a gasket of tarred oakum to center the pipe and hold the joint filling compound, after which the space shall be filled flush with such special joint filler compound as may be approved by the Engineer.

39. The use of Portland or natural cement mortar will not be permitted.

40. The gasket shall not occupy more than three-quarters ($\frac{3}{4}$) of an inch in depth on pipe twelve (12) inches in diameter or smaller, nor more than one (1) inch in depth on larger pipe.

Joint compound.

41. Before laying any pipe, the Contractor will be required to demonstrate to the Engineers that the pipe joint compound will meet the necessary requirements.

42. On dry work an asphalt compound may be used which has been prepared from a crude natural asphalt containing not less than ninety-eight (98) per cent pure bitumen soluble in carbon disulphide. It shall have a melting point of not less than 225 degrees F. and not over 280 degrees F. in accordance with the standard test recommended by American Society of Civil Engineers. It shall firmly adhere to the pipe, be absolutely watertight under a pressure of ten (10) feet of water and remain pliable at 32 degrees F.

43. On wet work a special joint filler compound prepared for the purpose or the sulphur and sand mixture herein described shall be used. If a special joint filler compound be used it shall be heated and poured in accordance with the special instructions of the manufacture.

Sulphur joints.

44. If a sulphur mixture be used for filling joints it shall consist of one (1) volume of sulphur and one (1) volume fine sand.

45. The sand shall be clean, free from loam or other foreign material and of such a fineness as to make a freely running mixture when mixed with melted sulphur.

46. The mixture shall be heated to a temperature to make it freely fluid (not less than 240 degrees nor more than 270 degrees) and kept at that temperature until used. The joint shall first be caulked and then poured in the same manner as in an ordinary lead joint.

47. After the joint is run and while the pipe is still warm, dry and clean, the joint shall be coated with hot tar pitch.

48. The preparation of the sulphur and sand and the making of the joints shall be done by men skilled in the work.

Jointing.

49. The interior of the pipe shall be carefully cleaned of all dirt, cement, or superfluous materials, of every description as the work proceeds, for which purpose a disc mold or swab filling the entire bore of the pipe, and attached to a rope or road sufficiently long to pass two (2) joints from the end of the pipe last laid shall be continuously worked through as the laying of the pipe proceeds. In all cases, the open ends of pipes and branches shall be temporarily and securely closed with carefully fitted boards or stoppers to prevent any earth or other substance from entering. The Contractor shall make good all defects and remove any foreign matter which may have been left in or otherwise introduced into the pipe before the final acceptance of the work.

Future Inlets.

50. Branches shall be built into the sewer at points ordered by the Engineers, to provide entrance for future connections. At future connections the bell end of the tile shall be blanked off with tile covers. These shall be included in the unit price specified to be paid per linear foot for the items herein specified. The blanks shall fit neatly in the open bell of the pipe and shall be made tight with cement mortar composed of one (1) part Portland cement and two (2) parts sand, or a suitable asphalt or composition joint filler applied hot. The cost of making such joints, cutting and placing the tile shall be included in the price specified to be paid under the items where specified.

51. Six (6) inch wye branches shall be placed in the sewer as required. For these an additional price will be paid as specified in Items 7, 8, 9 & 10, in addition to the price specified to be paid per lineal foot of sewer under Items 3, 4, 5 & 6.

Watertightness.

52. Thoroughly watertight work is required in both house connections, manholes and street sewers. The Engineer will carefully test the sewers to determine the amount of leakage of ground water into sewers during wet weather. During very wet weather the total infiltration of ground water into the whole system including manholes shall not exceed five hundred (500) gallons per twenty-four (24) hours per mile of main street sewer, nor shall the infiltration into any section of the sewer system exceed one (1) gallon per twenty-four (24) hours per lineal foot of main street sewer. If during the work or after its completion leaks are discovered the openings must be caulked or otherwise stopped. Any defects discovered in the work at any time before its final acceptance shall be promptly

corrected even if it is necessary to excavate, remove and rebuild portions of the sewer or appurtenances.

Measurement.

53. No extra or customary measurements of any kind will be allowed in measuring the work under these specifications. The actual length measured on the center line of the pipe, whether straight or curved, shall be used in computing the lineal feet of house connections or pipe sewer to be paid for. No additional payment will be made for specials, bends, cutting or other material except as herein specified to be paid extra per piece per wyes under Items 7, 8, 9 & 10.

Payment.

54. The unit price specified to be paid per lineal foot of vitrified sewer pipe under Items 3, 4, 5 & 6 shall include the furnishing and placing of the tile pipe, specials, etc., and the making of joints, and complete, as specified, and the furnishing of all labor, tools, materials and appurtenances necessary to complete the work as specified.

CONCRETE MANHOLES ITEM 11.

Description.

58. The Contractor shall furnish and place complete standard brick and concrete manholes with cast iron covers, step rungs, concrete bottom and appurtenances as specified herein or shown on the drawings.

Excavation.

59. The excavation, backfilling, resurfacing of streets, restoration of sidewalks and crossings, care of structures, bracing and other details of like character are not to be included in the price specified to be paid per manhole, but are to be included in

the unit price specified to be paid for sewer excavation under Items 1 and 2.

Manholes.

60. The manholes shall be cylindrical in form, with an internal diameter of five (5) feet excepting that the upper five (5) feet shall be conical in shape, being reduced to two (2) feet internal diameter at the top. The walls shall be eight (8) inches thick, built of concrete below Whiting Datum and above this elevation shall be built of two (2) courses of sewer brick laid on end in perpendicular courses in cement mortar; except as hereinafter specified. The top of said manholes shall be decreased to two (2) feet internal diameter, being drawn in by means of header courses, the diameter being decreased uniformly for each course.

61. The manhole shall rest on a foundation of concrete as specified in paragraphs 73 to 77 inclusive, twelve (12) inches in thickness, circular in shape and having a diameter of seven (7) feet. The invert of the sewers through the manholes shall be built of concrete of sufficient thickness to bring the floor of the manhole level with the top of the sewers entering.

Laying.

72. All brick shall be thoroughly wetted immediately before being laid, and old brickwork shall be thoroughly cleaned. Each brick shall be laid separately in a full and close joint of mortar on its bed, end and side at one operation. All joints on face shall be trowel struck. The refuse mortar shall be scraped away and removed before it has time to harden. The outside of the brickwork shall be plastered completely with a coat of cement mortar three-eighths (3/8) inch thick.

Concrete.

73. Below Whiting Datum (shown on the profiles as elevation 100.0) the walls of manholes shall be constructed of concrete composed of one (1) part Portland cement, two (2) parts torpedo sand, and four (4) parts gravel which will pass a screen having one and one-half ($1\frac{1}{2}$) inch openings, and be retained on a screen having one-quarter ($\frac{1}{4}$) inch openings.

74. The outer form for concrete shall be made of matched lumber which, after framing, shall be coated with asphalt as specified under paragraph 42, and shall be left in place as part of the manhole. Lumber for this purpose shall be included in the price bid for manholes and will not be paid for as additional lumber as specified under paragraph 11.

Mixing.

75. The concrete shall be mixed by an approved machine or by hand with arrangements which will secure a suitable mixing of each batch of concrete. These arrangements shall provide for the correct measurement of each of the ingredients before placing in the mixer, the mixing of the ingredients dry and the introduction of a measured quantity of water at any stage of the process. The mixing shall be continued until every particle of ballast is completely covered with mortar. No concrete will be allowed to be put in place after its initial set has taken place, and no retempered concrete will be allowed to be used under any conditions. The mixtures shall be moderately wet, and no excessively wet nor very dry concrete will be permitted.

76. After mixing, the concrete shall be transported as rapidly as possible and deposited in place. It shall be carried up level along the whole length of the section under construction, and shall

be so placed so to avoid rehandling within the forms. It shall be joggled or rammed into place by light ramming. On the face of exposed work, it shall be thoroughly spaded to bring rich mortar to the face. No concrete shall be deposited under water, and water shall not be allowed to rise or flow over the masonry until the cement has properly set. Special care shall be taken to see that the concrete is placed solidly against the forms, so as to leave no voids and to this end the Contractor shall use a suitable tool in order to remove any air entrained within, the concrete and to force the ballast away from the forms, thus leaving a layer of mortar next thereto.

77. Should any voids or other defects be discovered in any part of the work when the forms are taken down or otherwise, the defective work shall be removed and the space refilled with suitable material in a proper manner at the expense of the Contractor.

Step rungs.

78. The Contractor shall furnish and place step rungs of seven-eighths (7/8) inch diameter wrought iron spaced about sixteen (16) inches center to center vertically, staggered. The rungs shall project a minimum of six (6) inches inside the manhole from the face of the brick, and shall be long enough to pass through the brick and be hooked over one (1) inch on the outside. They shall be eight (8) inches wide on the step. The rungs shall be made of genuine wrought iron, neatly bent to give two (2) supporting bars through the brick, and shall be painted by dipping in hot pitch and oil varnish before setting.

PUMP PIT ITEM 13.

Description.

91. Under this item the Contractor shall construct a steel—

caisson, waterproofed, brick pit of twelve (12) feet internal diameter extending from the floor line of the pump house to elevation 86.22 as shown on the profiles, plan and details.

Steel Shell.

92. The Contractor shall provide a shell of boiler iron or mild steel completely encircling the walls of the pit from a point at least fourteen (14) inches below the floor of the pit to elevation 110. This shell shall be ~~of~~ three-eighths (3/8) inch in thickness below the elevation 92.00 and one-quarter (¼) inch in thickness above elevation 92.00. The necessary braces, channels, angles, shoes and plates required to stiffen this shell shall be supplied so that the shell may be truly circular in form when complete in place.

Waterproofing.

93. The interior of the steel shell and the floor of the pit shall be waterproofed below elevation 92.00 with five ply felt and asphalt, above elevation 92.00 with three ply felt and asphalt.

94. First cover the shell with hot roofing cement or asphaltum as specified under paragraph 42 and over this apply the necessary thickness (5 or 3 as required), of wool roofing felt weighing not less than 15 pounds (single thickness) to the square of 100 feet to be smoothly and evenly laid and well cemented together the full width of the lap, not less than nine (9) inches between each layer, with asphaltum at the rate of one hundred pounds (100) to the square of 100 feet.

95. Great care must be taken to make the waterproofing tight and free from cracks.

96. Waterproofing is to extend in an unbroken layer over the entire interior surface of the shell and foundation as shown on the

plans and details.

Brick Walls.

97. Construct a twelve (12) inch brick wall from a point fourteen (14) inches below the floor of the pit to the floor of the pump house, as shown on the plans.

98. Brick and mortar to be of the quality and laid as specified under "Manholes", paragraphs 62 to 72 inclusive, as far as the same may apply.

99. Brickmasons must use care not to injure the waterproofing.

Beams.

100. Place the necessary beams for foundation of motors and for steady-bearings as shown on the plans.

Floor.

101. A foundation of concrete about 6" thick shall be laid upon the excavation and the waterproofing laid upon it.

102. Upon this construction shall be laid the floor of the pit which shall be 14" in thickness and its finished surface at an elevation of 86.22.

103. Concrete for this purpose shall consist of Portland cement one (1) part to sand three (3) parts and gravel six (6) parts, mixed and laid as specified under paragraphs 73 to 77 inclusive, as far as the same may apply.

Sewer Connection.

104. Stiffen the steel shell around opening for sewer with 2" x 2" x 5/16" angle iron and build "bull's eye" in the brick work. Provide one length (at least four feet) of 24" cast iron socket pipe to fit snugly in the opening, and caulked and jointed with

asphaltum.

Screen chamber.

105. Construct screen chamber of one-half inch square mild steel bars and angles as shown on the details. "Z" bars and angle guide to be fastened to wall of pit with eight (8) three-quarter ($\frac{3}{4}$) inch bolts set in the brickwork.

Piping.

106. Furnish and set up in place all necessary piping for connections from the pumps to outlet, including 6" and 12" pipe, bends, hangers and branches. All piping to be well jointed and securely fastened to beams or wall.

Outlet.

107. Furnish and set in place one 12" x 6" x 18" "Y" with the necessary wall casting for connection to the force main.

Valves.

108. Furnish and set in the pipe lines one 6" and one 12" gate valve, Clow Company make or equal, bronze mounted, double seat with heavy stems.

Shear gate.

109. Furnish and set in the manhole nearest the pit, one 24" Sluice Gate, equal to Clow Company "A-3567" so arranged to shut off the flow from the sewer system to the pump pit.

Payment.

109 A. The price specified to be paid under this item shall include the furnishing and setting of the pit, floor, screen chamber, piping, beams, gate valves and sluice gate, as may be required to put the pit in working order.

PUMP HOUSE ITEM 15.

Structural Iron.

153. All structural iron of every kind throughout the building including floor beams, lintels, angles, columns, etc., will be of weights, sizes and dimensions as marked and figured on the plans, all to be framed and constructed in a proper manner for the position in which it is placed, and all to be provided with proper anchors for securing the same to the walls of the building. All framing to be done with standard size angles and bolts. All beam angles, and channels must be of steel, made by the Bessemer or open hearth process and to be uniform in quality and all fabricated work to be riveted. The Contractor will prove two sets of shop drawings of structural, also of the ornamental iron work for approval before the work is started. Every beam or angle in the building to have at least eight (8) inches of bearing on walls.

Plates.

154. All beams where resting on the walls to be provided with bearing plates.

155. Furnish diamond checked steel plate over pump basin to be in sections and to be removable.

Erecting.

156. The Contractor must set all structural ornamental iron work of every kind about the building, all to be done in a first-class and workmanlike manner.

Painting.

157. All iron work of every kind to receive one shop coat of mineral paint, and one field coat.

Electric Wiring.

177. Wire the building for (5) 100 Watt electric outlets, lo-

cated as shown, the wiring to be installed in accordance with the rules and regulations of the City of Whiting, governing such work. All lights to be controlled by a switch conveniently located.

Tile Roof.

178. The pitched roofs to be covered with green, dull Glaze tile--Spanish Style, all to be laid in the best manner and made water-tight in every part of the roof. The tile to be Ludowici-Geladon Co. manufacture, and to include all necessary hip rolls, and starters, and tile finial, all complete. All tile at eaves to have closed ends. The Contractor to provide all necessary flashings, all of proper sizes and shapes to flash all parts of the roof, in a way to insure good work and as may be directed by the Engineer. The nailing of tiles to be done with two copper covered nails. All ridges and hips to be made water-tight by cementing with good elastic cement.

Felting.

179. The surface under tile roofs to be first covered with one thickness of heavy tarred paper weighing not less than twenty-two (22) pounds to the square, shingle lapped with the pitch of the roof, and will be tacked down to the roof with flange headed nails.

Payment.

180. The unit price to be paid for the pump house shall include all work and material necessary to complete the same in accordance with the plans and specifications, including all cleaning up and incidentals necessary to leave the house and grounds in proper shape for operation.

SEWAGE PUMPING EQUIPMENT ITEM 16.

Description.

181. The work consists in furnishing and erecting complete two

(2) pumping units, electrically driven, which are to operate against a head of twenty-one (21) feet of capacity respectively eight hundred thousand (800,000) and three million two hundred thousand (3,200,000) gallons per twenty-four (24) hours; equipped complete with electric motors, automatic starting and stopping apparatus adjusted to operate with float regulators to be installed for each pumping unit. The pumps and appurtenances are to be erected in the pumping house to be built under this contract at the site selected by the City of Whiting at or near the corner of 119th and Schrage Avenue. Included in this contract are the furnishing of the pumps, motors, wiring, float regulators, controlling devices and check valves, complete, and the erection of the same in place on the foundations, with the furnishing of all wiring, piping and necessary appurtenances to put the same in complete running order as specified herein.

Drawings.

182. The Contractor will furnish detailed plans, showing the dimensions of apparatus and general specifications covering materials and method of construction. It is hereby agreed that within thirty (30) days following the signing of this Contract, the Contractor will furnish the Engineer for his approval three (3) complete sets of working blueprints, showing the foundation plans, and the complete dimensioned lay-out of the pumping machinery, with all necessary details covering the construction and arrangement of shafting, bearings, oiling devices, float regulators, automatic starting apparatus, electric wiring and connections, with complete specifications therefor attached.

183. Under this item the Contractor shall furnish and place

the entire pumping equipment as specified herein, including pumps, motors, automatic starting and stopping apparatus, floats, float tubes, cast-iron pipe, oiling devices, and wiring equipment, complete, ready to run in the pump house.

184. The heads given include friction in pipe lines, suction bends, and static lift with an allowance for the check valve. The pumps are intended to start flooded and to run until the sump or well is emptied. At times both pumps may discharge at the same time. The level of the sewage will fluctuate during pumping about seven (7) feet, on an average, but may be allowed a minimum of fourteen (14) feet.

Centrifugal Pumps.

185. The centrifugal pumps shall be of a make in successful use for two (2) years, the Lea-Courtenay or equal, built to run on sewage which has passed through a bar screen with three-quarter ($\frac{3}{4}$) inch openings. The pumps proper shall be designed and installed to operate with the pump runners, approximately twenty two feet below the floor line. The motors shall be above the floor.

Fittings.

186. The pumps shall be bronze fitted throughout and with bronze glands, runners being suitably balanced to minimize the thrust. Large bearings shall be provided, with suitable oiling devices. Ball or roller bearings shall be provided at the level of the upper floor, to carry the weight of the moving parts, including the pump runner and shaft, and a suitable flexible coupling to connect to the motor and a Base bearing shall be provided on the pit floor. The shafting shall be cold-rolled steel, provided with couplings of ample strength and intermediate steady bearings spaced approximately every six (6) feet on centers, each provided with

grease cups of ample size.

Motors.

187. The motors shall be vertical motors of the open polyphase squirrel-cage induction type, the General Electric Co. or Westinghouse make or equal, to run on two hundred and twenty (220) volt alternating current, three (3) phase, sixty (60) cycle. The motors shall be of the standard speed, to run at not to exceed 860 R. P. M.

189. The motors shall each be set in a neat tripod sub-base bolted to the floor plate, and shall be connected with the pump shafting direct by a suitable flexible coupling. The floor plate shall be bolted to the floor beams, or otherwise secured.

Oiling Devices.

190. Suitable automatic oiling devices shall be installed for the bearings of the vertical motors and roller bearings operated from a central reservoir in each house, which shall have ample capacity for at least thirty-six (36) hours continuous service. An automatic pump and filter shall be supplied for the motor bearing, to provide circulation, and a suitable similar equipment for the pump bearings. Grease cups of ample size shall also be provided as required on the shafting and pump bearings and at other points.

Controlling Devices.

191. A suitable automatic compensator and controlling device, or self-starter, shall be provided for each pump and motor. This shall include a self-starter of the Sundh make, as furnished by the Sundh Electric Co., 122 S. Michigan Avenue, Chicago, or equal connected with a float regulator and provided with an overload release and automatic cut-out to self-starting position in case of stoppage of current. Each float regulator shall be connected with a special

float working in a float tube to be furnished and placed under this contract. The float regulators shall be so arranged that either motor can be set to pick up the load first. Should the water level continue to rise the second float regulator shall then be ready to start the second motor.

192. The Contractor shall supply for each pumping unit neat slate panels carried on an angle iron frame and standards to be set on the floor. On the panel for each unit shall be mounted a self-starter as described on page 58 of the Sundh catalog or equal, equipped with a protective relay and an I-T-E made by the Cutter Co. or equal, time limit overhead circuit breaker of the two (2) coil three (3) pole type with rigid arms to work in unison, the starter to be energized by a switch controlled by the float. Each pump shall have a three (3) pole knife switch not fused, and there shall be a single pole knife switch so wired that the pumps can be started by hand. At the point where the feed wires are brought to the panel there shall be a main line three (3) pole knife switch, with fuses placed on back of panel, and space shall be left for watt meter to be furnished by the City.

193. The entire apparatus shall be of substantial construction, designed to afford accurate self-starting and stopping, with suitable protective features as specified to prevent burn-outs of the motor due to introduction of the current single phase, to release under sustained overload, under attempted starting under improper running conditions or closure under overload.

194. All electrical equipment shall conform to the requirements of the National Board of Fire Underwriters.

Floats and Float Tubes.

195. The Contractor shall furnish and place all floats, and float tubes required for the automatic operation of the motors as specified herein. The floats shall be of ample size, substantially built of solid copper suitably connected to the controlling devices by heavy copper cables and counterweights. The float tubes shall be of cast-iron of good quality or suitable weight, neatly and strongly supported and held in place by cast-iron or steel brackets or clamps, in a manner approved by the Engineer. All float tubes and brackets shall be thoroughly painted after erection with two (2) coats of an approved asphaltum varnish.

Wrenches and Oilers.

196. A complete set of wrenches shall be provided to fit all nuts and glands, and a neat oiler shall be provided for oiling.

Electrical Wiring.

197. The Contractor shall furnish and install all the electric wiring, and appurtenances necessary to connect up the pump motors, starting devices, float regulators, and other appurtenances in the pump house, to the main feed wire at its entrance to the pump house.

Check Valves.

199. The Contractor shall furnish two (2) check valves of suitable design, with ample waterways of dimensions to lay in the discharge pipe lines. The check valves shall be swing checks of the Clow Co. standard make or equal, or of other pattern approved by the Engineer. The body shall be of sound cast-iron of suitable quality. The facings of valve disc and seat and the pins shall be of suitable bronze composition machined to fit. The check valves shall be furnished with flanged ends, faced and drilled, ready to

lay, to the drilling ordered by the Engineer.

High Water Alarm.

200. The Contractor shall furnish and erect in the pump house and pit an additional float switch as specified for the starting apparatus, with float, cable, tube, and switch, to connect with an alarm gong in the fire engine house of the City of Whiting; an eight (8) inch gong with the necessary apparatus furnished and installed in said engine house to be operated from the float switch, connected to the batteries of the fire-alarm system; furnish and erect the necessary #14 duplex weatherproof wire upon the City's pole lines to connect up and operate the gong from the pump pit.

Erection.

201. The Contractor shall deliver all the pumping equipment and appurtenances herein specified on the appropriate site and shall furnish all the necessary labor, tools, materials and appurtenances to erect said pumping equipment and put it in running order for the official test.

Test.

202. When the Contractor has erected the pumping equipment and has completed the work he shall notify the Engineer, so that an official test can be made extending over four (4) hours actual continuous running time, under the conditions set forth in paragraph 184, as near as the same can be attained. The water pumped will be measured by means approved by the Engineer. The pressures against which the pumps works shall be taken from the pressure gages at the pumps, plus an allowance for the suction lift equal to the actual depth of the water level below the center of the pressure gage, plus an allowance for friction in the suction pipe, if

any, computed from Hazen-Williams Hydraulic Tables.

203. Should the flow of sewage or water be insufficient for the four (4) hours continuous test, four (4) tests of one (1) hour duration will be made at as close intervals as possible, or the equivalent in tests of shorter duration. Tests shall also be made of the floats and starting apparatus to demonstrate its efficiency, accuracy and reliability.

204. The electrical input shall be measured by a watt-meter in the pump-house.

205. The City will endeavor to have available the sewage, force main and other equipment necessary to test the pumps in actual pumping into the sewerage system when the work is completed. If arrangements are not then ready the Contractor shall send a representative to supervise the tests when notified by the Engineer, at such time as the test can be made.

Guarantee.

206. The Contractor guarantees that the pumping equipment he proposes to furnish will have an efficiency over all, including motor and pump, of:

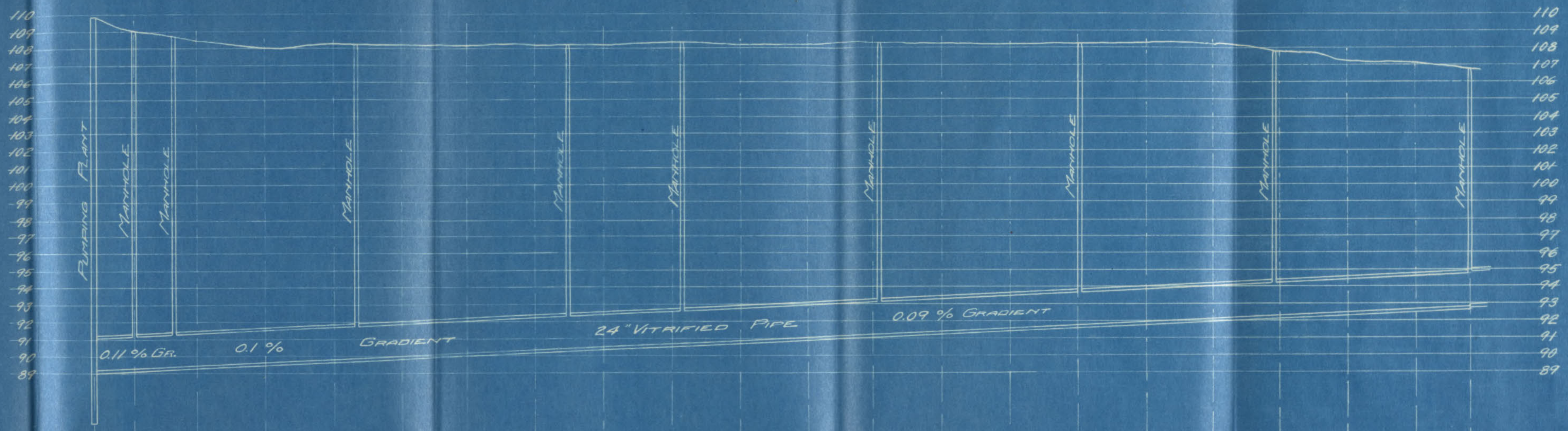
____________________ per cent of 800,000 gallon per day unit.

____________________ per cent of 3,200,000 gallon per day unit.

207. The Contractor also guarantees that he will replace or repair any deficiencies or breakage in the equipment installed which may be due to defective material, workmanship or design free of charge to the city, during a period of one (1) year after the final acceptance of the completed work after test.

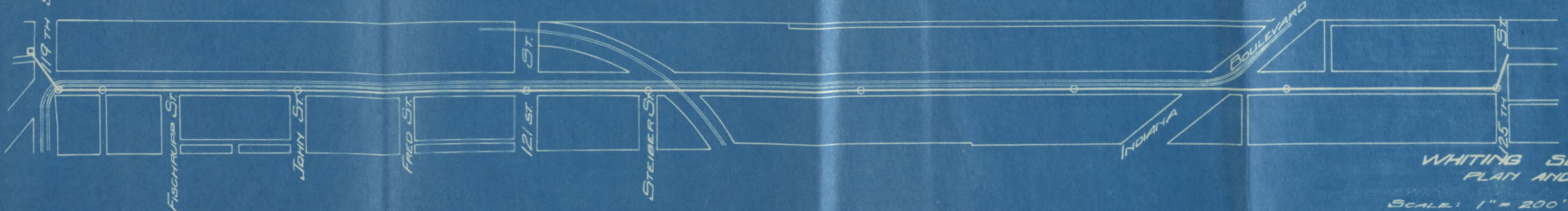
Payment.

208. The work for which the lump sums specified under Item 16 are to be paid for the pumping equipment and appurtenances complete erected in place shall include the furnishing and placing of the pumps, motors, wiring, electrical equipment, controlling devices, pressure gages, piping, check valves, wrenches, and miscellaneous apparatus, and shall include the furnishing of all labor, tools, material and apparatus necessary to complete the work as specified.



Station	13	14	16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50	52	53-33
Elevation	109.7	109.0	108.4	108.0	108.3	108.2	108.2	108.2	108.2	108.4	108.2	108.3	108.3	108.2	108.2	108.3	108.3	108.2	107.8	107.3	107.1	106.8
Grade	89.06	89.2	89.4	89.6	89.8	90.0	90.2	90.4	90.5	90.7	90.9	91.0	91.2	91.4	91.6	91.7	91.9	92.1	92.3	92.5	92.7	92.86
Cut	20.6	19.8	19.0	18.4	18.5	18.2	18.0	17.8	17.7	17.7	17.3	17.3	17.1	16.8	16.6	16.6	16.4	16.1	15.5	14.8	14.4	14.0

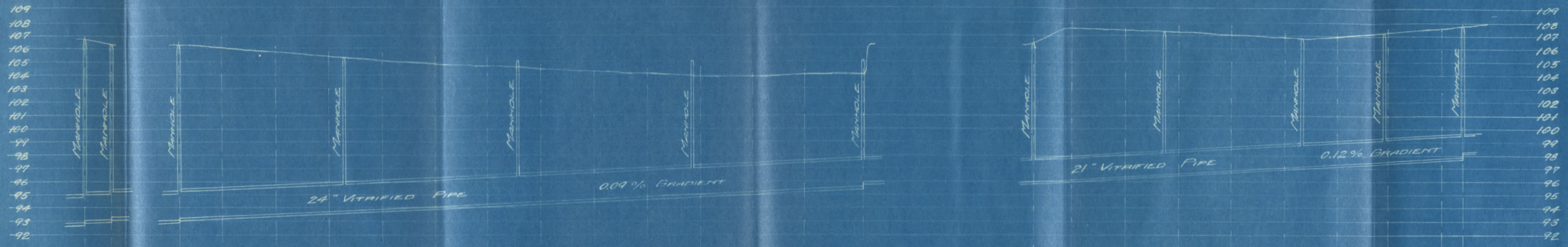
Schrage Avenue



WHITING SEWERAGE
PLAN AND PROFILE

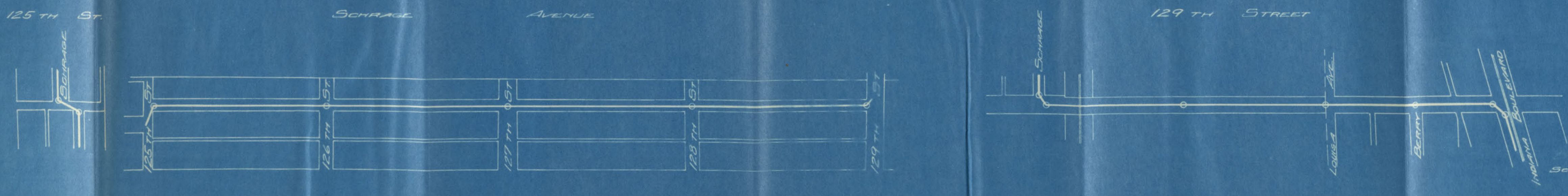
Scale: 1" = 200' Hor. 1" = 4' Vert.

1



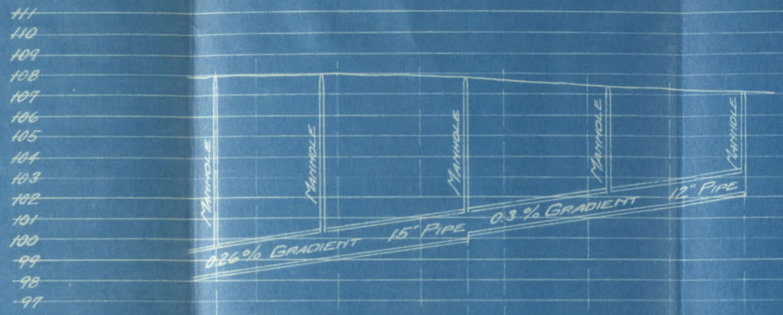
Station	53-33	54+30	54+30	56	58	60	62	64	66	68	70	72	74	76	78	80+14
Elevation	106.7	106.4	106.3	106.0	105.7	105.4	105.1	104.8	104.7	104.5	104.3	104.1	103.9	103.9	104.0	104.1
Grade	93.0	93.09	93.24	93.4	93.6	93.8	94.0	94.1	94.3	94.5	94.7	94.8	95.0	95.2	95.4	95.57
Cut	13.7	13.3	13.1	12.6	12.1	11.6	11.1	10.7	10.4	10.0	9.6	9.3	8.9	8.7	8.6	8.5

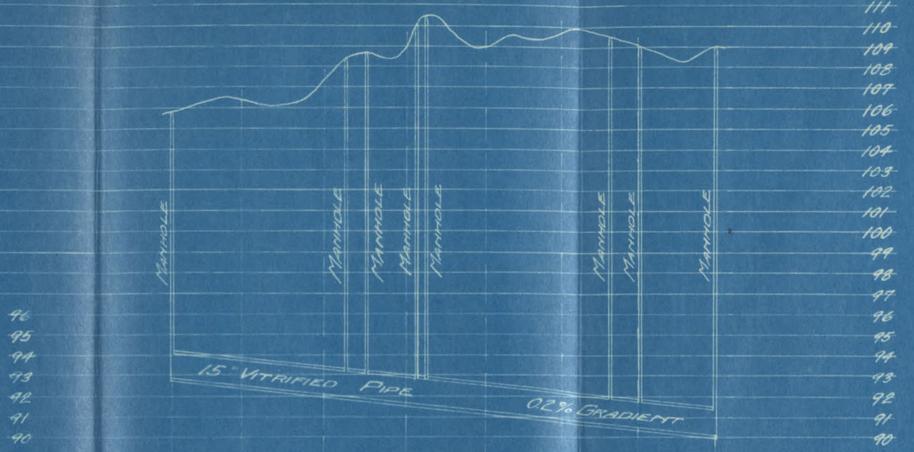
Station	80+51	82	84	86	88	90	92	94	96	96+62
Elevation	106.5	107.6	107.5	107.3	107.0	106.8	106.9	107.3	107.6	107.8
Grade	95.93	96.1	96.3	96.6	96.8	97.1	97.3	97.6	97.8	97.86
Cut	10.6	11.5	11.2	10.7	10.2	9.7	9.6	9.7	9.8	9.9



WHITING SEWERAGE
PLAN AND PROFILE

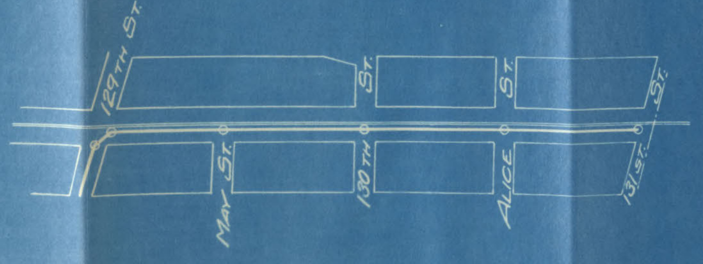
Scale: 1" = 200' Hor. 1" = 4' Vert.

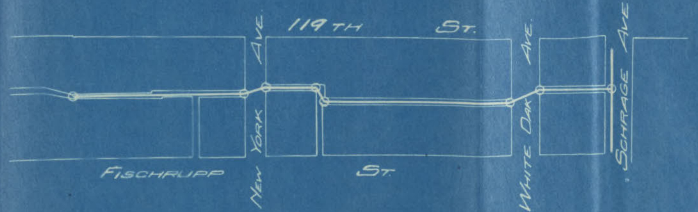




Station	97+01	98	100	102	103+61	104	106	108	110+15
Elevation	107.9	107.9	108.0	107.8	107.7	107.6	107.5	107.4	107.2
Grade	98.39	98.6	99.2	99.7	100.25	100.4	101.0	101.6	102.22
Cut	9.5	9.3	8.8	8.1	7.5	7.2	6.5	5.8	5.0

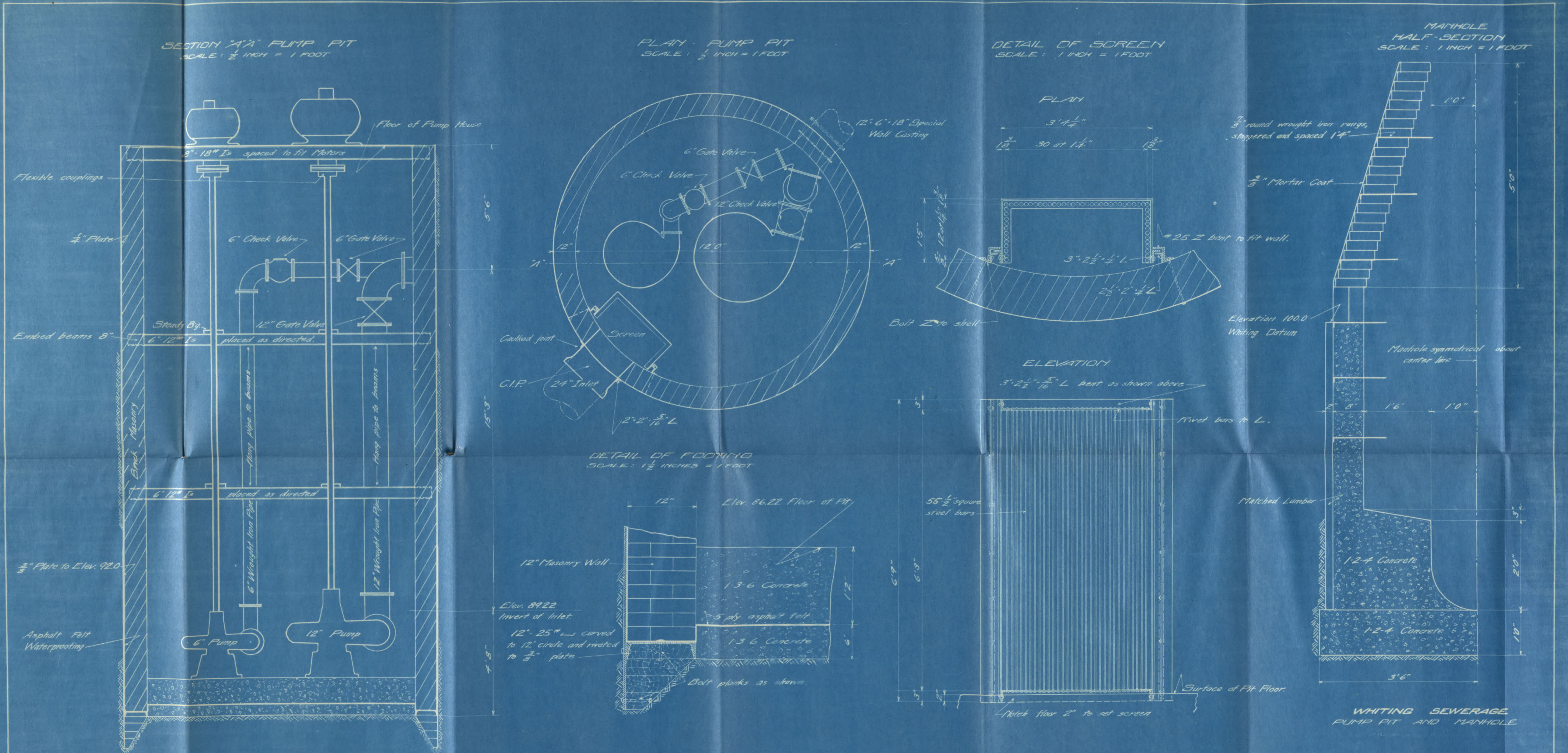
Station	13+75	12	10	8	6	4	2	0
Elevation	105.8	106.4	107.4	109.0	109.0	109.7	109.0	109.0
Grade	92.85	92.5	92.1	91.7	91.3	90.9	90.5	90.10
Cut	13.0	13.9	15.3	17.3	17.7	18.8	18.5	18.9

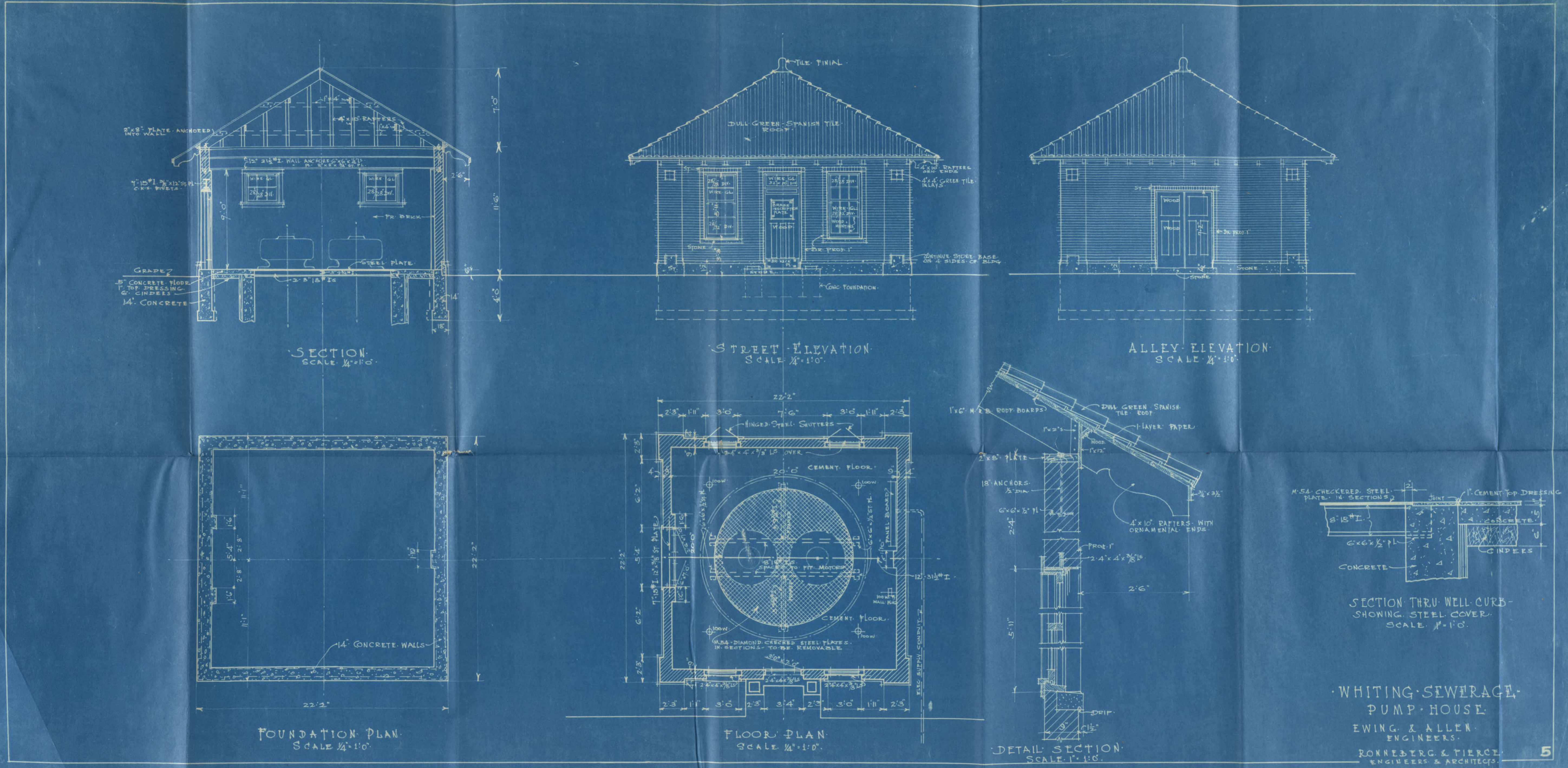




Whiting Sewerage
Plan and Profile

Scale: 1" = 200' Hor. 1" = 4' Vert.


SECTION "A-A" PUMP PIT
SCALE: 1/2 INCH = 1 FOOT
Floor of Pump House
5"-18" I's spaced to fit Motors
Flexible couplings
1/4" Plate
6" Check Valve
6" Gate Valve
Steady Bg.
12" Gate Valve
Embed beams 8"
6"-12" I's placed as directed.
Brick Masonry
Hang pipe to beams
6" Wrought Iron Pipe
12" Wrought Iron Pipe
6"-12" I's placed as directed
3/8" Plate to Elev. 92.0
Asphalt Felt Waterproofing
6" Pump
12" Pump
5'6"
15'3"
4'8"
PLAN - PUMP PIT
SCALE: 1/2 INCH = 1 FOOT
12"-6"-18" Special Wall Casting
6" Gate Valve
6" Check Valve
12" Check Valve
12"
12'0"
"A"
"A"
Screen
Caulked joint
C.I.P.
24" Inlet
2"-2"-5/16" L
DETAIL OF FOOTING
SCALE: 1 1/2 INCHES = 1 FOOT
12"
Elev. 86.22 Floor of Pit
12" Masonry Wall
1-3-6 Concrete
5 ply asphalt felt
1-3-6 Concrete
Elev. 89.22
Invert of Inlet
12"-25# curved to 12' circle and riveted to 3/8" plate
Bolt planks as shown
DETAIL OF SCREEN
SCALE: 1 INCH = 1 FOOT
PLAN
3'4 1/4"
1 3/8"
30 at 1 1/4"
1 3/8"
#25 Z bent to fit wall.
3"-2 1/2"-1/4" L
2 1/2"-2"-1/4" L
Bolt Z to shell
ELEVATION
3"-2 1/2"-5/16" L bent as shown above
Rivet bars to L.
55-1/2" square steel bars
6'9"
6'3"
Surface of Pit Floor.
Notch floor 2" to set screen
MANHOLE HALF-SECTION
SCALE: 1 INCH = 1 FOOT
1'0"
7/8" round wrought iron rungs, staggered and spaced 1'4"
3/8" Mortar Coat
5'0"
Elevation 100.0 Whiting Datum
Manhole symmetrical about center line
2" 8"
1'6"
1'0"
Matched Lumber
1-2-4 Concrete
1-2-4 Concrete
2'0"
10"
3'6"
WHITING SEWERAGE
PUMP PIT AND MANHOLE
4


SECTION
SCALE ¼"=1'-0"
STREET ELEVATION
SCALE ¼"=1'-0"
ALLEY ELEVATION
SCALE ¼"=1'-0"
FOUNDATION PLAN
SCALE ¼"=1'-0"
FLOOR PLAN
SCALE ¼"=1'-0"
DETAIL SECTION
SCALE 1"=1'-0"
SECTION THRU WELL CURB
SHOWING STEEL COVER
SCALE 1"=1'-0"
TILE FINIAL
DULL GREEN SPANISH TILE ROOF
2"x8" PLATE ANCHORED INTO WALL
4"x10" RAFTERS
GRADE
5" CONCRETE FLOOR
1" TOP DRESSING
6" CINDERS
14" CONCRETE
STEEL PLATE
PR. BRICK
CONC. FOUNDATION
CONTINUE STONE BASE ON 4 SIDES OF BLDG
STONE
WOOD
22'-2"
20'-0"
HINGED STEEL SHUTTERS
CEMENT FLOOR
PANEL BOARD
14" CONCRETE WALLS
M.54 DIAMOND CHECKED STEEL PLATES IN SECTIONS TO BE REMOVABLE
1"x6" M.&B. ROOF BOARDS
1 LAYER PAPER
2"x8" PLATE
18" ANCHORS ½" DIA
4"x10" RAFTERS WITH ORNAMENTAL ENDS
DRIP
M.54 CHECKERED STEEL PLATE IN SECTIONS
1" CEMENT TOP DRESSING
CONCRETE
CINDERS
JOINT
WHITING SEWERAGE PUMP HOUSE
EWING & ALLEN ENGINEERS
RONNEBERG & PIERCE ENGINEERS & ARCHITECTS
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